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THE WATER-WORKS OF PORTLAND, ORE.

By W. P. HARDESTY,* C. E.

Portland is much the oldest of the larger cities in the Pacific Northwest. It is located at the head of tidewater navigation on the Willamette River, about ten miles above its junction with the Columbia River and over 100 miles from the ocean. The place was first settled over 60 years ago.

The main city, including most of the business district, has always been on the west side of the river, and Portland proper was confined to this side until 1891, when East Portland was annexed. Owing to the plentiful precipitation west of the Cascade Range there is no difficulty in obtaining a plentiful supply of water for the city.

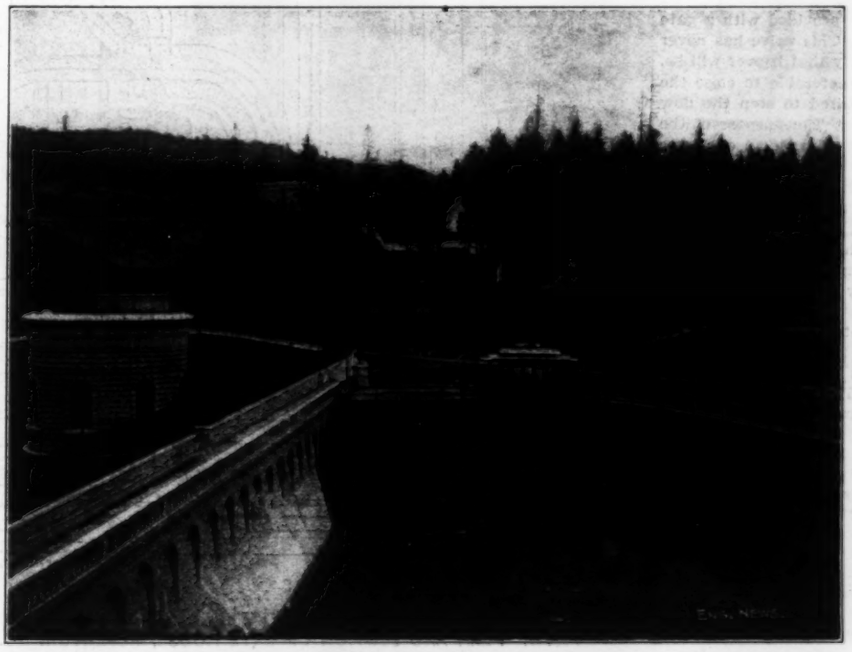
EARLIER WORKS.—The first water supply system was begun in 1857. Water from a small creek was diverted into wooden pipe made of fir logs by boring a 2½-in. hole from end to end by hand. A small steam pump was soon added to the plant. In 1861 about 5,000 ft. of new pipe was laid, made of redwood logs brought from California. In 1862 the Portland Water Co. was incorporated. In 1868 was begun the construction of the first pumping plant to take water from the Willamette River. This was located at the upper or south end of the city, but is now well within the city limits. Distribution reservoirs were used in connection with the pumping station, and in 1876 all were enlarged.

In 1883, to meet the needs of the growing city, the water company began the construction of an entirely new pumping plant. The point selected was on the west bank of the Willamette, about five miles above the center of the city. The location is at Palatine Hill, adjacent to the deep-water channel of the river. A large brick building was erected, in which were installed two 5,000,000-gal. pumps. A 30-in. wrought iron main was laid to a connection with the city distribution system. In 1891, five years after acquirement of the works by the city, an additional pump was installed, nearly doubling the capacity, and a 24-in. riveted steel main was laid to the city.

This pumping plant supplied the entire west side of the river until January, 1895, when the new gravity supply was brought in, since which time it has not been used but has been maintained in readiness to use at an hour's notice.

ACQUISITION BY CITY.—Early in the '80's agitation for municipal ownership began, and in 1885 an act was passed by the State Legislature authorizing the city to purchase or to construct and to operate an adequate system. At that time the estimated population was about 25,000, and the daily consumption of water about 4,000,000 gals.

In December, 1886, the entire system of the Portland Water Co. was purchased by the city. Since then the private plants of East Portland and smaller sections that have been annexed have been bought by the city.



VIEW IN CITY PARK, PORTLAND, ORE., SHOWING DAMS AND GATE HOUSES OF WATER-WORKS RESERVOIRS: NO. 4 IN LEFT FOREGROUND; NO. 3 ABOVE.

GRAVITY SUPPLY.—About the time the present pumping station was built, in 1883, attention was called to the desirability of obtaining a gravity supply. In 1885 a water committee of 15 prominent citizens of Portland was appointed. This committee selected Col. Isaac W. Smith, then living at Tacoma, Wash., as chief engineer, and he entered upon his duties in January, 1886.

Bull Run River was chosen as a new source of supply. A pipe line was located during 1886 and bids for new works were received, but construction was postponed for some seven years, owing to the purchase of the works of the Portland Water Co.

BULL RUN PIPE LINE.

Bull Run River heads in Bull Run Lake, a body of water covering probably two square miles lying near the summit of the Cascade Range and about ten miles northwest of the summit of Mt. Hood. The elevation here is about 3,500. After flowing westerly for nearly thirty miles, Bull Run joins the Sandy River, at an elevation of a little over 250. At seven miles above the junction is located the intake of the pipe line to be described.

In 1892 the "Bull Run Timber Reserve" was created. This tract contains 222 sq. mi., and it embraces the entire drainage area above the intake and an ample margin around it. There is not a habitation, pasture, road or even a trail in the drainage area. So rugged and wild is the country that it is with great difficulty that it can be penetrated on foot. The region is of volcanic origin, so that the water is soft. It is covered with dense timber and underbrush, excluding the sun. The water is of remarkable purity.

The pipe line follows near the right or northerly bank of the river for three or four miles; it then crosses to the south side, reaching the deep valley or canyon of the Sandy at six miles, which it crosses and follows down for three or four miles, after which it passes over the high ridge confining the valley on the west at Lusted's summit, and takes a more direct route for Portland. The general course is westerly. The line is really in the mountains for the first ten miles, and over a broken and rolling country for the remainder of the distance. For over one-half of the distance the pipe line was built over a right-of-way acquired for it, and for the rest it followed existing county roads. For about ten miles at the head the city was obliged to build a road, to make the country accessible at all and to provide for hauling material and supplies. Everything was hauled by wagon from Portland, and this formed a large factor in the cost of construction. In places the road follows the right of way, and in other places near to it. As it was planned to lay a second pipe alongside the first one, sometime, provision for this was made by laying the first one to one side of the center of bridges and trestles, and the deeper cuts were taken out the final width required for both lines.

INTAKE.—The diversion consists of a canal 15 ft. wide by 3 ft. deep and about 400 ft. long, with rubble masonry sides. The bed of the river at the head of this was blasted out so as to make a freer entrance. The excess of water in the canal spills over the lower wall. At the lower end is

*401 Seventh St., Portland, Ore.

placed a screen house, fitted with a rack screen and mesh screens, and also with gates for the regulation of the water. The elevation of the surface referred to true sea level is about 720.

FIRST DIVISION OF PIPE LINE.—This division is 10.02 miles long, reaching from the head to Lusted's summit, and has a hydraulic grade line with a fall of 6.26 ft. per mi.

The pipe is 42 in. in diameter. Three thicknesses of steel plate were used, No. 6 B. W. G. (.203-in.) for heads not exceeding about 150 ft.; No. 4 (.238-in.) for heads from 150 to 250 or 275 ft.; and ⅜-in. for heads over 275 ft. All pipe is double-riveted, and it was well coated with asphalt.

The pipe is made of plates having a net width of 5 ft., made up into 25-ft. sections. The sections were ordinarily joined by riveting, but in some places, where the excavation was in rock, sleeve and lead joints were used—in order to avoid the excavation required for riveting.

At all the summits and at abrupt changes in grade air valves were placed, and at the bottom of every depression a 6-in. blow-off valve was placed, to drain the pipe when desired. Manholes were placed at intervals of about 500 ft., their chief purpose being for convenience during the laying of the pipe.

About 28,000 ft. of pipe is under a pressure of over 50 lbs., over one mile of it having a pressure exceeding 100 lbs. In crossing the Sandy River valley nearly one-half mile of ⅜-in. pipe is used. The maximum head here is about 420 ft.

Horizontal bends and the larger vertical bends are made by special curve sections, a turn of about six degrees being effected at each one of the radial joints.

At Lusted's summit, at the junction with the next division of pipe, a 30-in. stand-pipe is placed on the main, reaching to a height of 25 or 30 ft. Just below this the main is provided with a gate valve, and a blow-off pipe. This valve has never been used, and it is not likely that it ever will be, as in practice it is found preferable to close the intake gates when it is desired to stop the flow to the city for a short time. The purpose of the stand-pipe is to allow an overflow if an excessive pressure should, from any cause, be brought on the pipe at this point. The pipe separates the different hydraulic grades used on divisions one and two.

SECOND DIVISION.—This portion of the line extends from Lusted's summit to Grants Butte summit, 7.77 miles. The hydraulic grade line has a fall of 14.64 ft. per mi.

The pipe is 35 ins. diameter, all made of No. 6 plate. Over 27,000 ft. of the line is under a pressure of over 50 lbs., on 7,000 ft. of which the pressure is over 100 lbs. The maximum head is about 275 ft. At the summit of Grant's Butte is placed another stand-pipe, at the junction with the third division. At the base of the butte, on either side, the head on the pipe is about 275 ft.

THIRD DIVISION.—This stretch of pipe line reaches from Grant's Butte to the Mt. Tabor upper reservoir, a distance of 6.51 miles. The hydraulic grade line has a fall of 19.75 ft. per mile, and 33-in. pipe is used. Practically all of the pipe is under a pressure of above 65 lbs., and nearly three miles of it has a pressure of over 100 lbs. The maximum head is about 275 ft. The pipe is made of No. 6 plate.

The total length of line from the intake to Mt. Tabor reservoir is 24.30 miles, and the total grade fall is 308.6 ft.

FOURTH DIVISION.—This taps Reservoir No. 1, into which the last division empties, and continues on west to the city. The pipe is 32-in. cast-iron, and the hydraulic grade is 16.67 ft. per mi. On nearly all of it the pressure is considerably over 100 lbs. The division is 4.06 miles long, terminating at the east side of the Willamette.

FIFTH DIVISION.—This is used in making the crossing of the Willamette River. The line is 0.38-mile long. There are two lines of pipe, a 28-in. cast-iron and a 24-in. welded steel pipe, the latter having been added for safety about four years after the first one was laid.

The pipe is submerged, and it was laid in the following manner. A trench was excavated for each pipe to a depth of from 8 to 16 ft. by means of a ladder-and-bucket dredge. The two pipe lines were put about 100 ft. apart. The cast-iron pipe was in lengths of about 17 feet, with ball and socket joints. The welded steel pipe was also provided with cast-iron joints, about one-half of the joints being made flexible. All joints were lead-calked. The pipe was laid from a timber cradle or inclined platform supported between two scows, the cradle reaching from the bottom of the river to over 20 ft. above its surface. The job was successfully done, and in only a few instances was it found necessary to employ a diver to calk the joints in the pipe after it reached the bed of the river.

SIXTH DIVISION.—This portion of the line reaches from the river to the upper one of the two reservoirs in the City Park, in the west part of the city. The length is 1.82 miles, and the hydraulic grade is 16.67 ft. per mi. The pipe is 32-in. cast-iron. Divisions four, five and six (herein arbitrarily assumed) really form only one division.

ENTIRE PIPE LINE.—The total length of pipe line from Bull Run intake to the City Park is 30.5 miles. If the water were not run into Reservoir No. 1 but came straight through the length of passage would be reduced to 30 miles.

The erection of the stand-pipes at the Lusted and Grant's Butte summits makes practically independent of each other the first three divisions of hydraulic grade line. The size of [illegible] for each division was proportioned to give [illegible] desired discharge of 22,500,000 gals., using [illegible]ter's formula with value of "n" equal to .013. [illegible]tual use shows that the first division (42-in. [illegible]ipe) has a capacity less than the second one, [illegible] the water does not entirely fill the pipe at [illegible]ted's summit; the discharge, therefore, indic[illegible] the total capacity of the first division but no[illegible] the second one. The second division also ha[illegible] less capacity than the third one, as the wat[illegible] does not entirely fill the pipe at the Grant's Butt[illegible] summit. The full capacity is required only [illegible] the first reservoir. The different capacities of divisions one, two and three is accounted for by the fact that division one has about 2½ times the total amount of angle in bends that No. 2 has and about five times that in No. 3, and hence offers more resistance to flow.

Soon after completion a test was made of the capacity of the pipe by observing the discharge into reservoir No. 1 for 12 hours. The time taken to add 1,000,000 gals. to the reservoir (indicated by a gage in the latter graduated to millions of gallons) was found to vary from 61 to 62 minutes. The daily discharge so measured, 23,500,000 gals., was compared with that indicated by the weir gage, 23,250,000 gals. It has been found that the discharge varies somewhat, sometimes reducing 2% or more, probably due to accumulations of air at the summits of the pipe line.

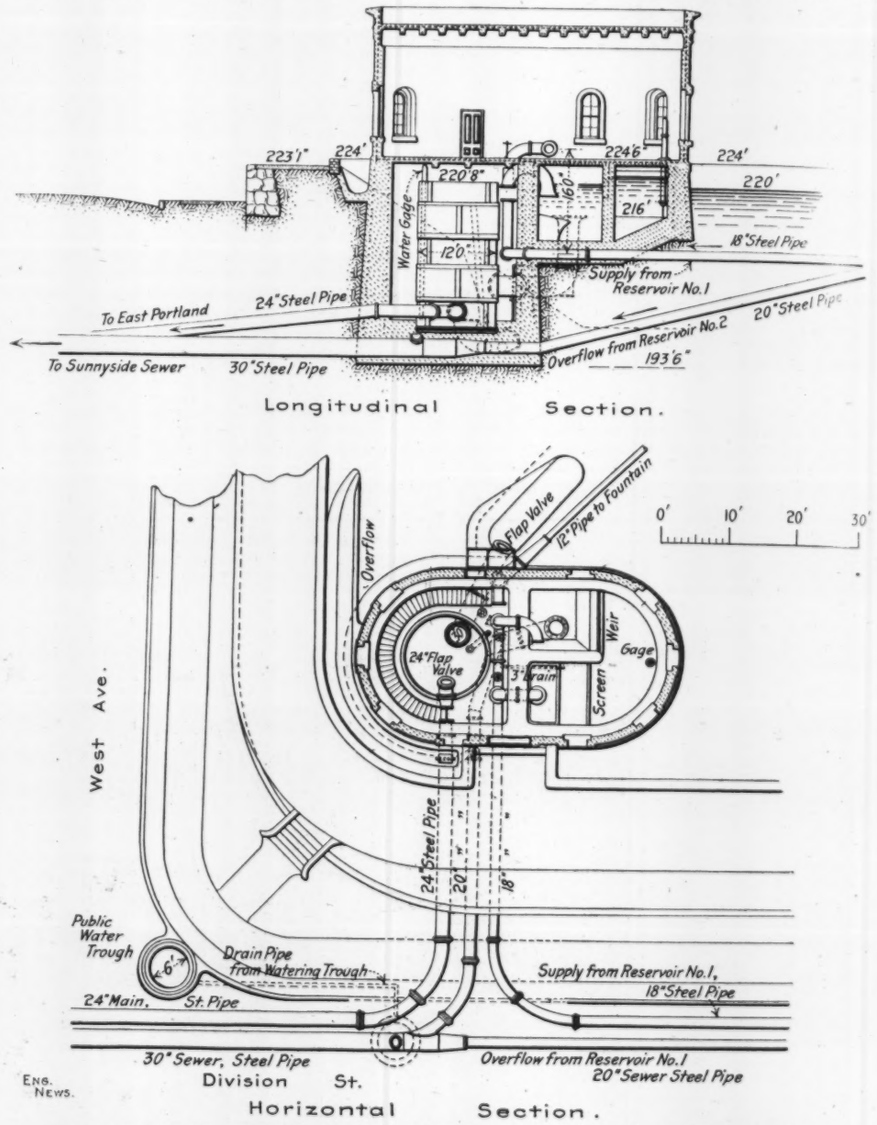


FIG. 1. PLAN AND SECTION OF GATE HOUSE, RESERVOIR NO. 2, WATER-WORKS OF PORTLAND, ORE.

RESERVOIR SYSTEM.

There are four reservoirs. Nos. 1 and 2 are on the east side and Nos. 3 and 4 on the west side of the Willamette River. The gate-chambers of these reservoirs were, at the time, of unusual design.

RESERVOIR NO. 1.—This is the highest reservoir of all. It is located on the south slope of Mt. Tabor, a low mountain about four miles east of the Willamette and near the east boundary of the city. The surface of the reservoir is about 410 ft. above sea level, the latter being about 5 ft. below low water in the Willamette at Portland. The reservoir was formed by constructing a dam across a large natural depression, using material excavated from it. The surface is about 330 ft. long by 280 ft. wide at its lower end. The maximum depth is 33 ft., and the capacity is 12,053,500 gals.

The dam is of concrete, with earth backing, and has 6-ft. top and 17-ft. bottom width. Its bottom is 2 ft. below the bottom of the reservoir, and a cut-off wall is carried 8 ft. lower still. On the face next to the water an earth apron is placed, whose top is a 10-ft. berme 22½ ft. below the water surface, from which a 1½ to 1 slope leads to the bottom of the reservoir. The backing is simply a fill made on the natural ground surface and reaching above the water line.

The sides of the reservoir were trimmed to a 1½ to 1 slope. The sides and bottom were lined with a layer of 4 to 6 ins. of concrete, laid in sections about 10 ft. wide, with joints between filled with asphalt. The surface was also coated with asphalt. The concrete was reinforced by being laid on a Ransome wire netting.

Regulation of water is made in a gate-chamber (see Fig. 1 for plan and section of gate-chamber of Reservoir 2, which is similar to this one). This is a concrete structure, set in the reservoir and adjoining the dam. It is oval in shape, about 24 by 44 ft., and is divided into two compartments by a wall across the middle. The concrete is reinforced with twisted bars of the Ransome patent. Water entering through the pipe from Bull Run empties into a concrete basin, whence it passes through a double set of screens and thence over a rectangular measuring weir. This weir is 7 ft. long. A float on the water surface actuates an indicator on a gage in the room above, one side of which is graduated into hundredths of a foot for weir depth and the other into millions of gallons corresponding.

From the weir the water passes to the other compartment, where it enters a steel tank about 12 ft. in diameter and 37 ft. deep. In this tank are the main regulating works. From it lead an 18-in. steel main to Reservoir No. 2, and the 32-in. cast-iron pipe that goes on to the city. Each of the pipe-ends projecting into the tank is provided with a flap valve, while outside of the tank is placed a gate valve, so that the outflow can be controlled by either one.

In the bottom of the tank is the opening by which the movement of water from the tank to the reservoir itself is regulated. This is provided with a cylinder gate, a cylinder reaching to above the top of the tank and raised or lowered by a cable passing over overhead pulleys to a hand-power hoist; most of the weight, however, is carried by a large counter-weight with the same cable connection.

The pipe connection between the tank and the reservoir reaches through the wall of the gate-chamber. Its outer end is provided with a flap valve. This is placed at the bottom of the reservoir; to prevent drawing from so low, however, when sediment and muddy water might be drawn into the pipe, the entrance is provided with a separate small open basin. Water in the reservoir can not be drawn below the top of the wall of this (about 3 ft. high) unless a butterfly valve in it be opened to admit the remainder of it to the basin.

At the top of the steel tank is placed an overflow, connected with a vertical pipe that descends just outside the tank and discharges into the sewer connection. In one corner of the reservoir

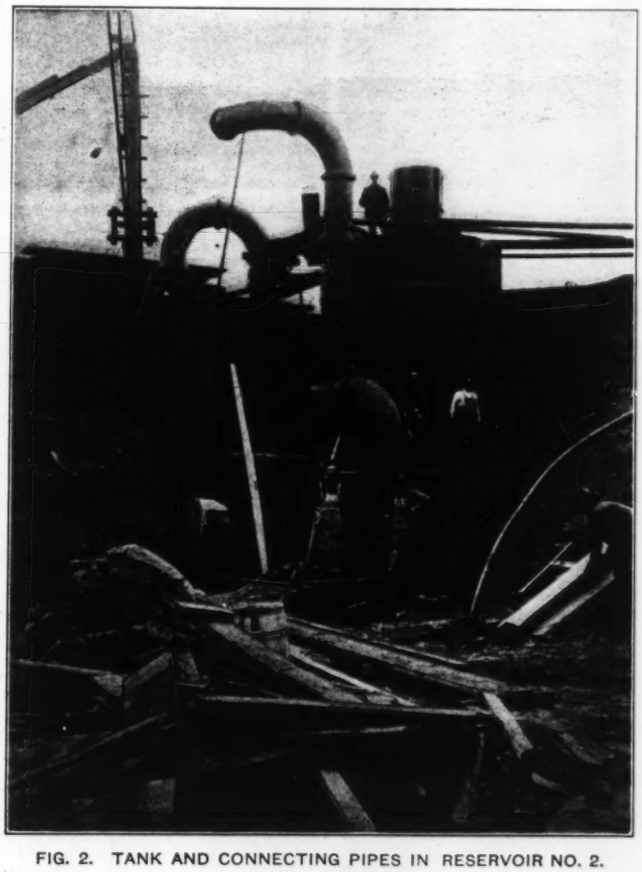


FIG. 2. TANK AND CONNECTING PIPES IN RESERVOIR NO. 2.

(The pipe with the compound curve is for connection with the proposed water-power plant.)

is located an overflow spillway, also leading to the sewer connection. The surface of water in the tank is practically at the level of that in the reservoir, and the excess in the reservoir can be disposed of by either of the two overflows. To discharge the excess water in this reservoir (and also in No. 2) into the city sewer system required the building of a 30-in. pipe for a distance of nearly two miles and of a 20-in. pipe for ½ mile.

RESERVOIR NO. 2.—From Reservoir No. 1 the 18-in. pipe before noted carries water down to No. 2, a distance of nearly ½ mile southwesterly. The difference in elevation is 182.4 ft.

Reservoir No. 2 is located on nearly flat ground at the northeast corner of West Avenue and Section Line Road. The reservoir is rectangular in plan, 693 ft. by 255 ft. The maximum depth is 20.4 ft., and the capacity is 20,533,500 gals. The basin is nearly all in excavation and embankment. The sides are finished to a slope of 1½ to 1, and they and the bottom are lined with two layers of brick, laid flatways in asphalt. This lining has not proved to be nearly so satisfactory as concrete, which was used on the other reservoirs.

At the southwest corner of the basin is the gate-chamber (Fig. 1). The general plan and arrangement of this are the same as for the other reservoir.

It was intended to utilize the fall from Reservoir No. 1 to this reservoir to develop power for pumping water to some of the high-lying districts around Mt. Tabor, and a pipe and connection in the chamber room were provided for this purpose (Fig. 2), but nothing further has been done.

RESERVOIR NO. 3.—This is the upper one of the two reservoirs in City Park (Figs. 3 and 4), in the west part of the city. The basin occupies a natural depression, a large gulch, and was formed by building a masonry dam. The basin has a surface elevation of about 300. The greatest depth is 49 ft. and the capacity is 16,450,600 gals. The reservoir sides are trimmed to a slope of 1½ to 1, and the sides and bottom are lined with concrete.

The dam is concrete, with its exposed face finished in cement to give the appearance of cement-block ashlar masonry. It is curved up-stream in plan, having a radius of 400 ft. The thickness on top, on which there is a roadway and two parapet walls, is 15½ ft.; at the highest place the height is about 53 ft. and the bottom width is 41 ft., under which is a 7-ft. cut-off wall. The outside or face is quite ornate, with blind arches near the top. The length is about 180 ft.

The picturesque surroundings afforded by the City Park make this a most attractive part of the water-supply system of Portland.

A pumping plant is a part of Reservoir No. 3. In it are utilized the fall to Reservoir No. 4, it being located at the upper end of the latter. There are two single-acting Riedler pumps, provided with differential pipes, that take water directly from feed pipes connected with the main from Reservoir No. 3. The head obtained is 63 ft. Each pump is direct-connected to a 6-ft. Pelton wheel, the normal speed of which is 106 r. p. m. The pump stroke is 16 ins. One of the pumps is used to lift water to Portland Heights, a district with elevations ranging from 500 to 750. The water is pumped against a head of nearly 600 ft., through a 6-in. special Converse lock-joint pipe, which connects with two 50,000-gal. redwood tanks. This pump has a 9-in. plunger, and the capacity is 500,000 gals. per day.

The other pump supplies water to an intermediate high service district, elevation from 250 to 500, reaching from Willamette Heights on the north to a point in South Portland, a distance of about six miles. The head pumped against is nearly 300 ft. The pump has a 11½-in. plunger, and a capacity of about 1,000,000 gals. per day.

All the water passing from the upper to the lower reservoir can be used for power. During times of full use in the districts supplied by the pumps, the consumption for this power amounts to about 15,000,000 gals. daily. As this is more than required by the district supplied through the lower reservoir, some of the water is wasted.

To avoid this waste, there has lately been installed another pump, driven by electric power, This is a Gould's single-acting triplex plunger pump, with normal speed of 36 strokes per min. It is driven by a 100-HP. G. E. motor, with 5:1 chain-drive connection, and by proper proportioning of gear wheels the reduction from the 700 r.p.m. speed of the motor is made. The capacity for pumping to the Portland Heights district is over 500,000 gals. per day.

RESERVOIR NO. 4.—This reservoir (Fig. 4) is located at about the mouth of the same hollow that No. 3 is built in. It has a maximum depth of 40.5 ft., and has a capacity of 17,711,800 gals. The elevation of the surface is about 230.

To form the basin required the building of a long wall and dam on the east side. The dam is of concrete, of the same general design but somewhat less height than that for No. 3, and it is straight instead of being arched in plan. The length is about 230 ft.

SLIDES OF RESERVOIR SLOPES.—The hillsides against which Reservoirs 3 and 4 were built proved to be very much given to sliding, and it was found before construction had advanced very

far that they would have to be well-drained. This provision did not prevent extensive slides after their completion, requiring much expensive work for repairs and prevention of further slides. These land-slides have been very thoroughly described by Mr. D. D. Clarke, M. Am. Soc. C. E., in Trans. Am. Soc. C. E., Vol. 53, p. 322 (1904), in an interesting paper with discussion, so they will be merely mentioned here.

Repairs to the reservoir linings were completed only about two years ago. For the most part they were made by putting a second layer of concrete on the old one. Between the two was placed Hydrex felt, in three thicknesses on the west side of the reservoirs, where the greatest displacement had occurred, and in two thicknesses on the east side. The different layers of felt were laid with asphalt between.

CONTRACT PRICES.

The Bull Run work was let in several different contracts, being: (1) Headworks; (2) Supply of steel plates; (3) Manufacturing and laying steel pipe, and the excavation and backfilling of trench, furnishing and placing of specials, etc.; (4) Furnishing and laying cast-iron pipe; (5) Steel bridges and trestles; (6) Furnishing and laying submerged cast-iron pipe; (7) Excavation for reservoirs.

Prices of some of the principal items were:

(2) Steel plates, delivered at Portland, 2.98 cts. per lb., they having been manufactured in Pennsylvania.

(3) Manufacture and laying of the different sizes of pipe, 2.75 cts. per lb.; manholes, $20; air valves and blow-off valves, $20; excavation and back-filling, earth, 30 cts., loose rock, $1.50, solid rock, $3; concrete, $8.

(4) Cast-iron pipe, $32 per ton; laying of same (including lead, etc.), $5 per ton.

(5) Steel bridges, 6 cts. per lb. for steel in place.

(6) Submerged pipe across the Willamette, $31.43 per lin. ft.; laying of pipe, $2.50 per lin. ft.; dredging, 40 cts. per cu. yd.

(7) Reservoir excavation, earth 30 cts.; loose rock, 45 cts.; solid rock, $1.20; hardpan, 40 cts.

The contracts for construction of the pipe line were let in March, 1893, and the work was completed in December, 1894. The excavation for reservoirs was begun late in 1893, and all reservoir work except the excavation was done by day labor.

TOTAL COST OF BULL RUN SYSTEM.

Expenditures on the Bull Run supply began as early as 1886, when nearly $8,000 was spent on surveys and explorations, and over $7,000 on the headworks. The detailed and total expenditures on this supply to the close of 1896 (representing original works) and 1906, respectively, are given in the accompanying table.

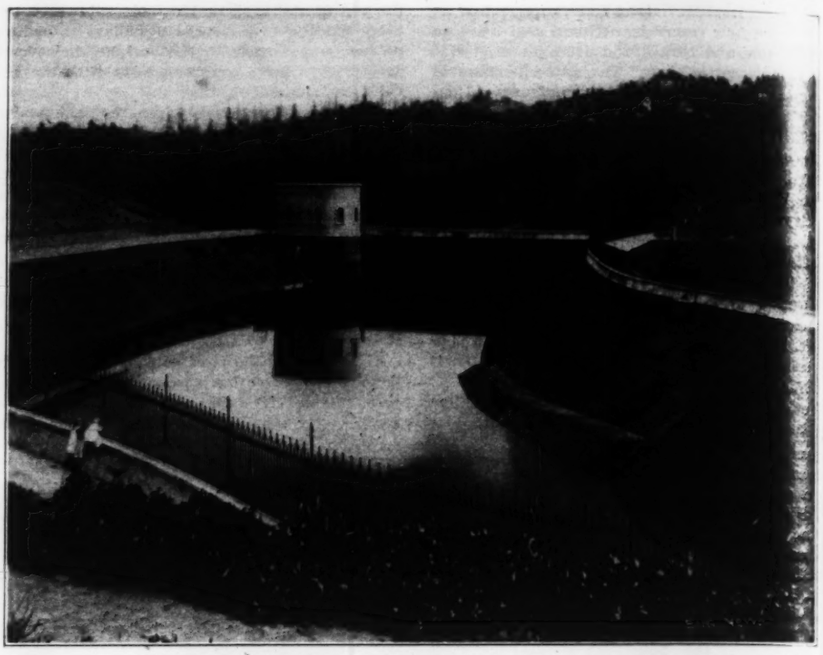


FIG. 3. VIEW OF RESERVOIR NO. 3, LOOKING TOWARDS DAM.

GENERAL.

There are two principal zones or levels to be supplied in the distributary system of Portland. The low service reaches from the level of the Willamette to about elevation 150; the so-called high service reaches from about elevation 150 to 250; and above the latter are scattered districts, some of them reaching to over 800 in elevation. Some of these have already been noted. The several suburban cities that have been annexed to Portland have had water-works owned by private corporations, and these have been bought by the city of Portland. In 1902 was purchased the pumping plant of the Albina Light & Water Co., and this is still used to supply the higher levels of Albina and other annexed districts on the "Peninsula" north of East Portland.

During the season of maximum use, the warmest days of the summer months, about 23,500,000 gals. per day are brought in by gravity, and about 4,000,000 gals. are pumped from artesian wells. The minimum discharge of the Bull Run River is about 65,000,000 gals.

The combined storage capacity of the four reservoirs is 66,750,000 gals., equivalent to three days supply from Bull Run.

The old pumps at the Palatine Hill station are kept only for use in case of stoppage, caused by a break or otherwise, in the Bull Run supply. They can be started up at an hour's notice, at any time. Occasionally the machinery is turned for a few hours to keep everything in good working order. But they have never been called upon since the Bull Run water was turned into the pipes, on Jan. 2, 1895. Practically all distribution is by gravity, as the pumped supply is first run into storage tanks.

The following statement of mileage of pipe 4 ins. and over will give an idea of the growth of the system: Purchased in 1886 with the original Portland water-works, 27.4 miles; since then, 62.3; total of mains purchased, 89.7 miles; total mains in use at end of 1906, 216.3 miles. In addition to the 4-in. mains there is a sufficient length of 3 to 1-in. pipe in service (including 24.6 miles of 2-in.) to bring the total length of the distributing system up to 251.9 miles. Of this grand total, 187 miles are of 30 to 2-in. cast-iron pipe and 65 miles are of 30 to 1-in. steel or wrought iron pipe, the nearly 25 miles of 2-in. pipe being in the latter class. The 31 miles of the Bull Run supply mains and reservoir connections brings the total pipe length up to 283 miles.

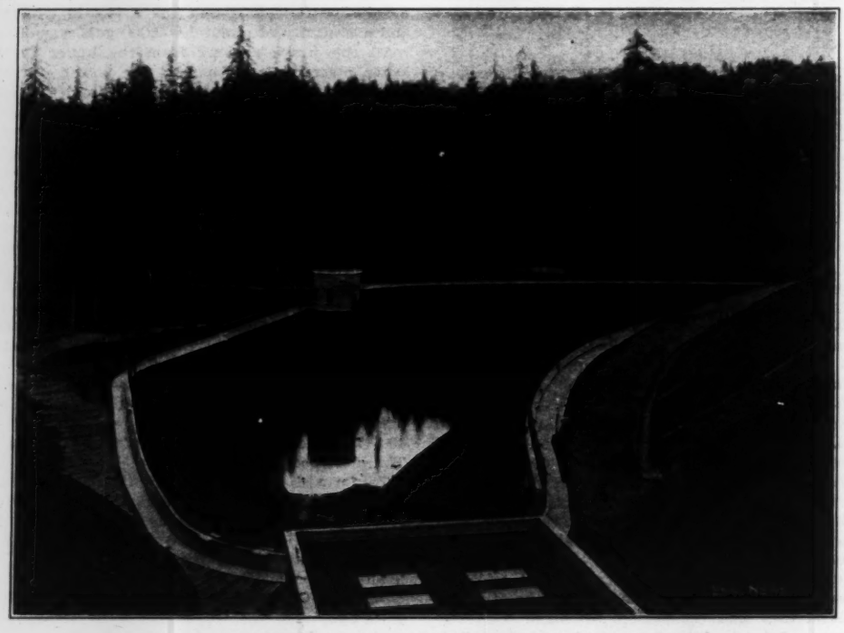


FIG. 4. RESERVOIR NO. 4, LOOKING DOWN FROM RESERVOIR NO. 3.
(Roof of power-house in foreground; slope protection against sliding shown on the right.)

TABLE: COST OF WORKS.

	1896.	1906.
Surveys and explorations	$9,688	$9,688
Lands, riparian rights and rights-of-way	47,666	61,742
Clearing and roads	33,046	33,046
Headworks	57,246	57,246
Steel plates for pipe, 24.3 miles	377,312	377,312
Manufacturing and laying same, trench work, etc.	569,946	569,946
Bridges for pipe across Bull Run and Sandy Rivers	54,896	57,146
Cast-iron pipe and laying of same, Mt. Tabor to City Park, 5.9 mi.	288,589	288,589
Submerged pipes across Willamette River	78,798	132,652*
Reservoirs (4), including land	813,697	1,140,994
Portland Heights supply, water pumps, etc.	30,897	31,477
Telephone line to headworks, 30 mi.	7,262	7,262
General expenses	61,463	85,116†
Total	$2,430,507	$2,851,215

*The second submerged pipe (of welded steel) was not laid until 1898.

†The general expense account, including maintenance of central office, etc., ends with 1902, when the water committee was succeeded by a water board, as provided by the new charter of Portland, which went into effect Jan. 1, 1903.

Outside of the Bull Run supply, the city of Portland has spent the following:

Portland Water Co.'s works (in 1886), $464,552; other private works purchased from 1897 to 1906, $258,170; making a total of $722,722; additional pumps and mains for Palatine station, $274,486, making a total cost of all the pumping systems purchased and improved as above, $997,208. Distribution system, extending mains, $1,356,231 to end of 1906. Total cost of the whole water-works system to end of 1906, $5,204,654.

WORKINGS OF MUNICIPAL OWNERSHIP.—The city of Portland has owned and operated the water-works since Jan. 1, 1887. The water rates have been reduced several times, and are now about one-third those charged by the private corporation which formerly supplied water to the city. The gravity system was substituted for pumping on Jan. 2, 1905.

During the eight years that the city pumped water from the Willamette River the operating expenses and repairs amounted to 33% of the cash receipts; during the 12 years ending with 1906, with a gravity supply, they amounted to but 13.7%, notwithstanding the reduction of rates meanwhile. During 1906, the operating expenses and repairs amounted to 14% of the cash receipts, and the interest on the $2,900,000 of bonds to 28.5% and the sinking fund 5.2%, leaving 52.3% net profit available for extensions of the water-works.

CONSTRUCTING ENGINEERS.—Most of the water-works construction of Portland has been carried out under charge of Col. Isaac W. Smith, who was Chief Engineer of the Water Committee from Jan. 1, 1886, to the date of his death, on Jan. 1, 1897. Colonel Smith was a veteran in the engineering work of the Pacific Northwest, having first come to this region in 1854. He was a member of the American Society of Civil Engineers.

Mr. James D. Schuyler, M. Am. Soc. C. E., was Consulting Engineer for construction of the reservoirs and the submerged pipe across the Willamette River. Mr. David D. Clarke, M. Am. Soc. C. E., was Principal Assistant Engineer in charge of construction of the pipe line. Mr. Clarke is Engineer of the Water Board of Portland, and to him the writer is indebted for information from which this article has been prepared.

At a bond election held in Portland in June, 1907, it was voted to issue $3,000,000 to build a second pipe line from Bull Run to the city, and to improve the distribution system. At the time that the present pipe line was built the design provided for a duplicate main alongside the one then built, as has already been noted.

A LOGARITHMIC DIAGRAM FOR THE FLOW OF WATER IN OPEN CHANNELS.

By GEORGE T. PRINCE,* M. Am. Soc. C. E.

In Engineering News of July 6, 1899, there appeared a logarithmic diagram relating to the flow of water in clean cast-iron pipes under pressure which I had prepared after the design of A. Van Muyden, C. E., which appeared in your issue of Sept. 22, 1898. I have been considerably surprised and much gratified to learn how generally this diagram has been used, and because of this fact I am sending you a diagram similarly prepared which relates to the flow of water in open channels.

It will be observed that the diagram is based upon the Chezy formula, the value of "C" in that formula being assumed as 100. Velocities in channels having other values of "C" can be obtained from the diagram by simple proportion.

The determination of the value of the coefficient is dependent upon judgment born of experience, and even then it is largely a matter of guess work.

One familiar with irrigation problems or the flow of water in open conduits can as readily guess the value of "C" in the Chezy formula as the value of "N" in the Kutter formula.

If precision is desired by one not so experienced it would be better to first ascertain the value of "C" by the Kutter formula and proportion the velocity as given by the diagram accordingly.

*Chief Engineer, The Denver Union Water Co.

THE DESIGN OF RAILWAY REPAIR SHOP PLANTS.*

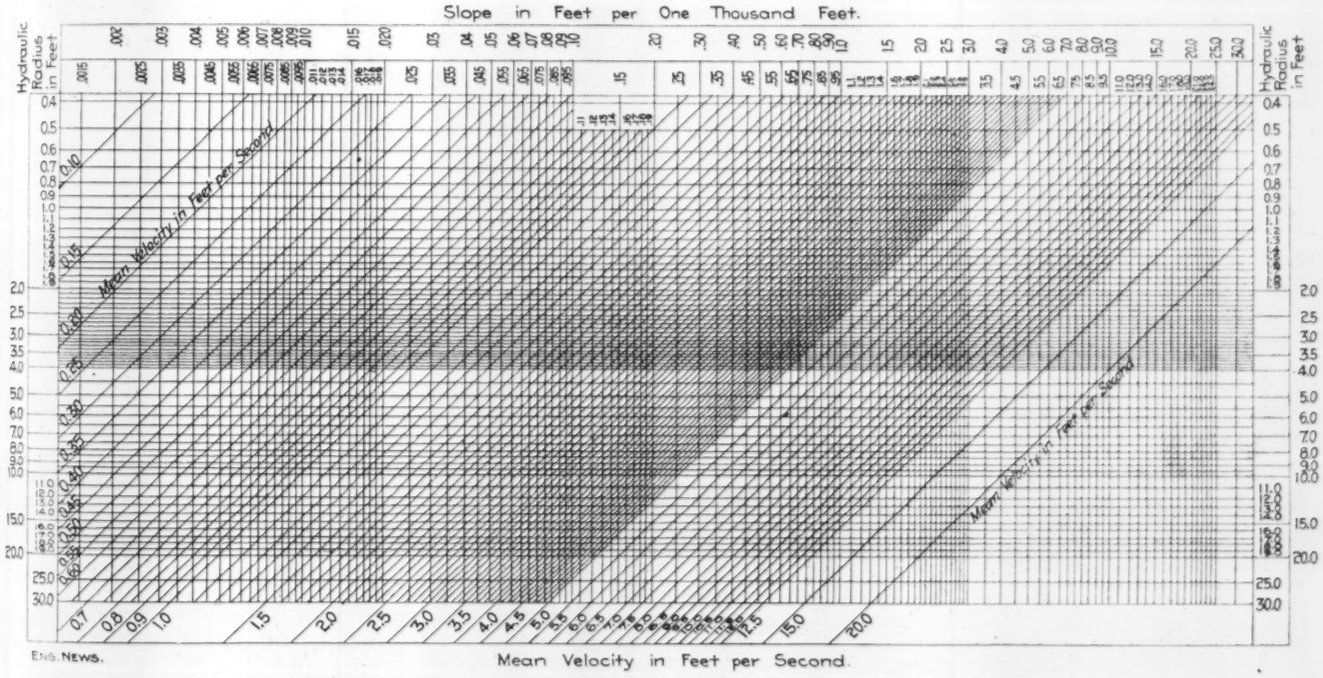
By GEORGE A. DAMON.†

The original cost of an average modern locomotive may vary from $14,000 to $17,000. The actual expense for labor and material to maintain this locomotive in first-class condition will range from $1,500 to $3,000 per year, each road, of course, having its own individual conditions that influence this cost. For our present purposes, however, it may be said that a locomotive costing $15,000 will involve a maintenance cost of about $2,250 per annum, and this expenditure should go far toward overcoming the natural depreciation of and wear on the equipment, and thus keep the locomotive in first-class running order.

The original investment in a complete new locomotive repair shop, fully equipped to efficiently make these repairs, will depend on the location, the size, the design, the selection of the equipment and the cost of labor and material entering into the buildings. These factors are so variable that it is impossible to determine an average that can be regarded as an exact standard. Even an approximate estimate for any given case can only be obtained after a detailed study has been made of the conditions surrounding the particular problem in hand. The following figures must, therefore, be used with judgment, or they may be misleading. These figures have been taken from the records of the actual cost of a number of repair shops with the design and construction of which the writer has been connected. The fact that there is such a wide variation emphasizes the danger of deriving a general law from any particular experience, or even of drawing a conclusion for a particular case from a general experience.

Our records show that locomotive repair shops which are laid out on a basis of the number of pits required equal to 6% of the number of locomotives served can be built and equipped complete for an expenditure ranging between $50,000 and $65,000 per pit. If one pit will serve 16.66 engines per year, the cost of repair facilities will fall between the limits of from $3,000 to $4,000 per locomotive. An investment amounting to the lower limit is absolutely necessary if the engines are to be kept on the road. Just how much more than this should be spent in order to secure the minimum cost of locomotive maintenance, including all the items of not only actual repair expenses, labor and material, but also interest, depreciation, insurance and taxes upon the plant

*Extracts from a paper read before the Canadian Railway Club, Jan. 7, 1908.
†Managing Engineer, The Arnold Co., Chicago.


Slope in Feet per One Thousand Feet.
Hydraulic Radius in Feet
Mean Velocity in Feet per Second
Mean Velocity in Feet per Second
ENG. NEWS.

Mean Velocity in Feet per Second.

LOGARITHMIC DIAGRAM RELATING TO THE FLOW OF WATER IN OPEN CHANNELS.

Geo. T. Prince, M. Am. Soc. C. E.

The "C" in the expression $v = c\sqrt{rs}$ is assumed to have a value of 100.

EXPLANATION.—The Vertical Lines indicate the Slope of Channel per 1,000 ft. of Length. The Horizontal Lines indicate the Hydraulic Radius $= \frac{\text{area of channel}}{\text{wetted perimeter}}$.

The Inclined Lines indicate the Mean Velocity of the Flow in Feet per Second.

EXAMPLES.—A channel having R = 3.5, has a slope of 0.7 ft. per 1,000 ft. What is the mean velocity of flow? Follow along the horizontal line marked 3.5 to its intersection with the vertical line marked 0.7; the inclined line at the point of intersection indicates the mean velocity to be 4.9 ft. per sec.

A channel has a slope of 0.40 ft. per 1,000 ft., and the mean velocity of flow is 4 ft. per sec. What is the value of R? Follow down the vertical line marked 0.40 to its intersection with the inclined line indicating a velocity of 4 ft. The point of intersection falls on a horizontal line indicating R = 4.0.

Total flow of channel in cubic feet per second equals area of cross-section in square feet multiplied by the mean velocity of the flow in feet per second. To find the mean velocity of the flow when "C" has some other value than 100, multiply the mean velocity as indicated by the diagram by the ratio, $\frac{\text{value of "C"}}{100}$.

provided, is a question that should have the most careful consideration.

The analysis of the total cost given in Table No. 1, will serve to indicate the relative importance of the decisions that must be reached in order to give each dollar expended a maximum earning capacity. The sum total of the "low" and of the "high" figures shown will result in grand totals which will show a wider range than the 33 1-3% variation indicated by the unit figures of $3,000 to $4,000 per locomotive, but as it is improbable that any shop would be built using either the lowest or the highest estimate for every one of its parts it will be found that only in exceptional cases will the actual total cost fall outside of the limits first given.

TABLE I.—COST LIMITS FOR LOCOMOTIVE REPAIR SHOPS ON THE BASIS OF 25 ERECTING PITS.*

Shop Yards.	Limits of Cost Per Pit "Low."	"High."	Approx. Proportion of Total Cost. P. ct.
Tracks, crane runways, transfer or turn tables	$1,400	$3,000	4
Water and sewer systems	1,000	1,800	2
Piping and wiring tunnels and tunnel piping	500	1,000	1
Buildings.			
Machine and erecting shop	8,000	12,000	17
Boiler and tank shop	3,000	5,000	7
Forge shop	1,500	2,400	4
Storehouse and offices	1,000	2,500	3
Locomotive carpenter shop	500	1,000	1
Power house	1,200	2,400	3
Oil house and equipment	400	600	1
Miscellaneous buildings; scrap bins, material sheds, fences	500	1,000	1
General Equipment.			
Power house equipment	5,000	8,000	11
Traveling cranes	1,500	3,000	4
Tool equipment	10,000	15,000	22
Heating system	1,200	2,500	3
Power and lighting systems including yard wiring and lighting	1,500	4,500	5
Plumbing and lockers	300	1,000	1
Air, water, steam and oil piping in buildings	600	1,200	2
Incidentals, organization and engineering	2,000	7,000	8
			100

*Note.—These figures do not include items for real estate and preparation of the shop site, which cost necessarily varies between wide limits. The foundry building and equipment are not included in these figures.

The history of "modern" locomotive repair shops may be considered as beginning with the use of what has been called "the electric drive." The use of the electric motor makes possible: (A) the arrangement of departments independent of the line shaft requirements; (B) the use of electric cranes; and (C) the construction of all power producing equipment in one central power plant from which may be distributed heat, water, compressed air, light, and power. The Oelwein shops of the Chicago Great Western Ry. were among the first to make use of the electric motor for both individual machines and groups of tools, as well as the electric crane and the central power plant. These shops were put in commission in 1900. Since then there have been over 70 shops either built entirely new or modernized from previously existing plants. It is safe to say that more than $80,000,000 has been expended upon railway repair shops and their equipment within the last ten years.

Some of the first shops designed to use a central electric power plant were handicapped by the use of the 220-volt direct-current system, which made it desirable to limit the radius of distribution to about 1,000 ft. from the generating plant. The development of the induction motor and the alternating system soon removed this limitation, but as it was still necessary to use the direct-current motor to obtain satisfactory electric speed control on many of the independently motor driven tools, a mixed electric system was adopted by many shops. It is still the practice in a number of recent installations to provide both direct and alternating current throughout the shop. Direct current is used for the cranes, variable speed motors, and perhaps the mercury vapor lamps; and alternating current for the constant speed motors, incandescent lights, and the yard lighting systems. The recent improvements in mechanical speed changing devices; the development of the alternating-current crane, and the reduction in cost of the induction motor are creating a decided tendency toward the elimination of the direct current system. By these improvements it is possible to distribute power from a central plant over a larger area and also remove the old limitations of the amount of ground covered by the shop plant.

The size of a locomotive repair shop will depend primarily upon the number of locomotives to be repaired each year, and the number of days each locomotive under repair must occupy an erecting shop pit. To the pits required for "heavy" repairs must be added a number of pits for "light" repairs, for emergency work, and for future growth. The ratio of the number of pits to the number of locomotives will vary with the assumptions as follows:

A = Number of locomotives served by shop.
B = Percentage of locomotives to undergo "heavy" repairs each year.
C = Average number of days each locomotive occupies an erecting shop pit.
D = Number of working days each year.
P = Number of pits required for "heavy" repairs.
P' = Number of pits required for "light" repairs.
P'' = Total number of pits required.

Then

$$P = \frac{A \times B \times C}{D} \text{ and } P'' = P' + P.$$

The effect of the different assumptions as to the percentage of the total number of locomotives to be repaired each year and the length of time on a pit are shown by Table II., which was prepared for a shop serving 400 locomotives, with a probable increase to 500 within the next five years. From this it will be seen that the number of pits may vary from 5% to 7% of the total number of locomotives, which variation corresponds with actual practice. Individual conditions will govern the assumptions for each road, and there are a number of other methods of checking these calculations.

TABLE II.—LOCOMOTIVE REPAIRS.

No. of Locos. A	Percent. Repd. B	Days in Shop. C	Working Days in Year. D	Pits, Heavy Repairs. P	Pits, Light Repairs. P'	Total Pits. P''	$\frac{P'}{P}$	$\frac{P''}{A}$
500	80%	20	288	28	7	35	25%	7%
500	70%	17	288	21	9	30	43	6%
500	60%	14	288	15	10	25	66	5%

The size of the machine shop space per pit depends largely upon the methods adopted for operating the shop. The work on the pit can be crowded, and the erecting output under normal conditions can be doubled during rush periods, but the output of the machine tools can not be increased in the same ratio. The tendency at present is to increase the amount of floor space allowed for machine tools. On the other hand, the use of high-speed tools is rapidly increasing the output of each machine, and this development is contributing to the output efficiency of each foot of machine tool space. The space per pit for machine tools varies from 1,500 to 2,500 sq. ft. This result is often largely influenced by the demands made upon this space for manufacturing new work, or for supplying repair parts to other shops.

An average rule for the size of the boiler shop is to allow one-third as much space for this department as is allotted to the machine and erecting shop, but this rule is affected by the character of boiler repairs to be made. The kind of coal, the conditions of water supply, and whether or not new boilers are to be constructed, must be known before a final decision can be reached. The blacksmith shop is ordinarily one-third the size of the machine and erecting shop. Local conditions must again be carefully considered before reaching conclusions. The safest way for all shops is to canvass the requirements for output and the capacity and floor space required of each machine, and then design the shop to fit the machines, rather than crowd or spread out the machines into a space the size of which has been too hurriedly determined.

TESTS OF THE BOND BETWEEN CONCRETE AND STEEL.

An addition to the already variable data upon the value of the bond between concrete and reinforcing steel has just been made by Prof. J. L. Van Ornum, Professor of Civil Engineering at Washington University, St. Louis, Mo., for the Reinforced Concrete Construction Co. of that city. The experiments consisted of a series of direct pulling tests upon steel rods embedded in 12 × 12-in. prismatic blocks of 90-day old 1:2:4 concrete. The bars used were plain round ones in two sets, one of medium steel with an elastic limit around 40,000 lbs. per sq. in., and the other of high carbon steel, elastic limit about 58,000 lbs. per sq. in. In each set one test was made on rods of the following diameters: ½-in., ⅝-in., ¾-in., 1-in. and 1¼-in. The rods were embedded for 25 times their diameter in the medium steel specimens and for 40 times the diameter in the high carbon steel.

The accompanying table gives all the data of the tests. In every case it will be noted the maximum pull on the embedded rod was higher than the elastic limit of the steel, which indicates that slipping did not commence until after the elastic limit of the steel was reached. After the bond had been broken and slipping had occurred there still existed an appreciable friction between the steel and concrete from 60 to 80% as great as the bonding value of the hardened concrete against the steel.

The tests cover only a small portion of the field; there was no determination of the effect of varying the length of embedded steel, no graded readings for various amounts of slips and no tests on any but plain rods. Still they are interesting to compare with other straight pull bond tests, which, as we have noted before in an editorial, show values in nine tests varying from 93 to 854 lbs., with an average of 515 lbs.

UNIFORM ACCOUNTING REPORTS for municipal industries and public service corporations are receiving a large amount of attention of late. The American Association of Public Accountants has appointed a Committee on Standard Schedules for Uniform Reports on Municipal Industries and Public Service Corporations. Mr. Harvey S. Chase, of 27 State St., Boston, Mass., is chairman of the committee.

TEMPORARY BRIDGES, which can be built of soldiers' knapsacks, are to be used as part of the equipment of the Mexican army. The knapsacks, which are an invention of Colonel of Engineers C. A. C. Lopez, are of wooden frame with light steel connections so arranged that they can be joined together to make a substantial truss some twelve ins. deep and wide enough for one man.

TESTS OF BOND BETWEEN CONCRETE AND STEEL.
Made at Washington University, by Prof. J. L. Van Ornum, M. Am. Soc. C. E.

		Physical properties of steel bars tested.						Bonding properties of steel bars tested. 11/29/07.				Friction after slipping had occurred	
Mark.	Size of tested section.	Maximum load producing rupture.	Maximum strength in lbs. per sq. in.	Limit of elasticity in lbs.	Limit of elasticity in lbs. per sq. in.	Elongation of tested section.	Per cent. of elongation.	Modulus of Elasticity.	Length of embedment in concrete.	Surface of embedment, sq. in.	Maximum pull in lbs.	Max. pull in lbs. per sq. in of embedded surface.	Friction in lbs.	Friction in lbs. per sq. in.
Medium steel bars.	½"	12,280	60,200	8,260	40,500	2.13"	26⅝	29,900,000	13.8"	21.7	10,050	460	7,500	340
	⅝"	18,510	57,500	12,640	39,300	2.30"	28¾	29,300,000	16.7"	32.8	14,010	430	10,100	310
	¾"	24,580	55,600	17,050	38,600	2.32"	29	29,800,000	20.6"	48.5	18,120	370	15,980	330
	1"	45,370	58,900	29,580	38,400	1.91"	23⅞	28,600,000	25.7"	80.7	31,110	390	30,510	380
	1¼"	75,350	60,900	48,140	38,900	1.80"	22½	29,800,000	32.3"	126.8	49,300	390	45,000	350
High carbon steel bars.	½"	18,710	88,300	12,280	57,900	1.26"	15¾	29,200,000	20.8"	32.7	13,120	400	6,800	210
	⅝"	27,580	85,700	17,440	54,200	1.46"	18¼	28,900,000	25.6"	50.3	18,380	370	12,000	240
	¾"	48,430	109.600	26,820	60,700	1.10"	13¾	30,600,000	32.5"	76.6	29,930	390	20,600	270
	1"	76,470	95,500	42,560	53,100	1.56"	19½	29,700,000	32.7"	102.7	48,260	470	29,000	280
	1¼"	101,330	83,900	65,940	54,600	1.66"	20¾	30,400,000	50.5"	198.3	66,140	330	39,320	200

JOHN GALBRAITH: PRESIDENT OF THE CANADIAN SOCIETY OF CIVIL ENGINEERS.

By J. G. G. KERRY,* M. Can. Soc. C. E.

Dr. John Galbraith, who was elected President of the Canadian Society of Civil Engineers at the annual meeting on Jan. 28, is one of the best known and most highly esteemed members of the profession in the Dominion. A Canadian by birth and education, he finished his college training by graduating in 1868 as Gold Medallist in Honour Mathematics and Prince's Prizeman at the University of Toronto; the Prince's prize being a special prize for highest general proficiency, established by the Prince of Wales, now King Edward VII., during his visit to Canada.

The Dominion of Canada having been confederated in 1867, Dr. Galbraith is one of that body of eminent men whose working life has been contemporaneous with that of the Dominion, and who with quiet and consistent patriotism have struggled for its upbuilding and have prospered with its growth. With the establishment of Confederation there came a great outburst of engineering activity all over the settled portions of Canada, and particularly in transportation works; and Dr. Galbraith on graduation found employment in the railroad field, getting his professional training as apprentice to Geo. A. Stewart, at that time Chief Engineer of the Midland Railway and also an engineer and surveyor of extensive private practice. Later, in 1871, after a year's service as contractor's engineer on the construction of the Intercolonial Railway, then being built by the Dominion Government as part of the Confederation program, Dr. Galbraith returned to the Midland Railway as Resident Engineer, afterwards becoming Division Engineer.

The Midland Railway was at that time being pushed westward to reach the Georgian Bay with all the enthusiasm and faith in the future that were characteristic of its promoter, the late Baron Hugel, who in spite of the great financial crisis then existing attained his end, though he failed to win the expected reward of his labors. It is noteworthy that both the great Canadian railways are to-day actively working on the development of the lake terminals first reached by the old Midland Railway during Dr. Galbraith's connection with it, and are perfecting their main eastbound lines for lake and rail business over the route then selected.

From 1875 to 1877 Dr. Galbraith was employed on surveys for the Canadian Pacific main lines, then under direct Government control, and for the projected Georgian Bay branch of that undertaking.

The Chief Engineer of the Georgian Bay branch was Mr. Wm. Murdoch and an associate of Dr. Galbraith's on this work was Mr. H. D. Lumsden, who is now Chief Engineer of the National Transcontinental Ry., and has been President of the Canadian Society of Civil Engineers. Dr. Galbraith and Mr. Lumsden held the positions of Division Engineers.

By reason of this experience in the Northern Canadian wilderness with that which he has obtained on many subsequent journeys, made both for professional purposes and for private enjoyment, through the great areas of unsettled bush land that completely cover the northern portions of the Provinces of Ontario and Quebec, there are to-day few men in Canada better qualified to equip and guide exploring parties; the best proof of this statement being the fact that Dr. Galbraith in the summer of 1881, unaccompanied and for recreation only, made a trip from Lake Superior via Michipicoten to Hudson's Bay, thence via Rupert's River to Lake Mistassini and across the height of land to Lake St. John, reaching civilization by way of the Saguenay River.

In 1878 Dr. Galbraith was appointed Professor of Engineering in the Ontario School of Practical Science, and since that date his work has been continuously identified with that of the school; this has recently been made the Engineering Faculty of the University of Toronto, a Provincial institution maintained by the Province of Ontario. To the development of the School of Practical Science, Dr. Galbraith has devoted the whole of his time and abilities, consistently refusing to undertake professional work as a consulting engineer whenever it was likely to interfere with his duties as a teacher. The School of Practical Science is to-day the largest and most rapidly growing engineering school in Canada. This, while in a measure due to the great prosperity of the Province of Ontario, must in large part be credited to the work that has been done by Dr. Galbraith; for the successive classes of graduates have recognized the thoroughness of their training, the simple devotion to duty of their professor, the soundness of his professional knowledge and the manliness of his personal character so fully that it is questionable whether any other engineering school on the continent gets such enthusiastic advertising and such active personal support from its own graduates. It is characteristic of the man, and of his estimate of his own attainments, that he has repeatedly refused permission for the publication of his lectures and has contributed but little to the sometimes too-swollen stream of

J. Galbraith

technical literature. Dr. Galbraith's work since 1878 may be aptly described as that of a fashioner of tools which have been widely used by other men in the construction of the great engineering works of Canada; the standing of the graduates of the School of Practical Science is evidence of the thoroughness of his fashioning.

Dr. Galbraith has occupied many honorary positions, including those of Vice-President of the Ontario Land Surveyors Association; Vice-President of the Engineering Section of the British Association for the Advancement of Science; Vice-President of the Engineering Section of the American Association for the Advancement of Science. He has received the degree of LL. D. both from Toronto University and from Queens University. He has also been consulted on many engineering questions by the Government of Ontario and by the City of Toronto, a most important instance being in connection with the real meaning of the power leases granted by the Government of Ontario at Niagara Falls.

When last summer the engineering world was startled by the fall of the Quebec Bridge, it was recognized in Canada that the disaster must be investigated by commissioners of unquestioned impartiality and integrity and of sound engineering knowledge, whose conclusions would be unhesitatingly received by the country at large, the undertaking having been practically a Government work. That Dr. Galbraith possessed the necessary qualifications to an eminent degree was everywhere recognized and his appointment to the Commission of Inquiry was immediate, the University of Toronto willingly releasing him from his professional duties for a whole session in recognition of the importance of the inquiry and in appreciation of his long and notable services to the University. It is fitting that his election to the Presidency of the Canadian Society of Civil Engineers should come just as the work of the Commission is drawing to a close and while the engineering world is anxiously awaiting a full and thorough discussion of the great disaster that is under investigation.

*Consulting Engineer [of the firm of Smith, Kerry & Chace, Toronto, Ont.].

NEW MACHINERY FOR THE APPLICATION OF ABRASIVES TO MARBLE WORKING.

By J. ROYDEN PEIRCE.*

Probably no class of manufacturing has been less affected by modern technical methods than the marble industry. The following description and accompanying illustrations show the general outlines of a new process which has succeeded in obviating the difficulties hitherto encountered in working this beautiful, but very fragile building material, and which produces work of a character which has never been obtained before.

The marble block, as it comes from the quarry, is sawed into slabs by means of gang saws. These consist of a series of parallel steel blades fastened in a heavy rocker frame; there are no teeth on the blades, but a constant supply of sand and water is fed to them. In this way, the marble is slowly abraded. It takes about a week to saw a large marble block into 7/8-in. slabs. The surface of these slabs or plates coming from the saws is very imperfect. Not only is it uneven, but the finish is rough.

The slab is now placed upon the first machine used in the new process—the drum rubber, shown in Fig. 1. A massive, cylindrical carborundum drum, 6½ ft. long and 16 ins. in diameter, revolving at 1,200 r. p. m., cuts the surface of the marble. Owing to the very fragile nature of the slabs, it is necessary to use great precautions to prevent their being fractured by vibration; consequently the drum is placed under the slabs and they are pressed upon it by a rubber-covered pressure roll. The weight of this roll is proportioned exactly to balance the upward thrust of the carborundum drum.

So successful has this arrangement been that the most delicate and fragile marbles are handled without breakage. The machine will rub 1,000 sq. ft. in 8 hours. It leaves a finish known technically as the "spine finish," on which, when held to the light, the slabs have the appearance of woven silk. This finish gives a peculiar, opalescent sheen to the marble. Furthermore, the coloration and veining of the marble are fully exposed, making it an easy task to match the slabs properly. The object of the rubber and, indeed, of any surfacing machine, is first to obtain a true plane; and secondly, to remove the marks left by the gang saw. Setters using marble from this machine claim to effect a very material saving over older processes, due to the trueness of the pieces of marble.

From the drum rubber, the slabs are placed upon the coping machines, where they are cut to the desired sizes. The cutting operation is performed by a thin carborundum wheel, 12 ins. in diameter and 3/8-in. thick. All of the machines are provided with a copious supply of water, which prevents the least trace of burning, notwithstanding the fact that the most powerful abrasive (carborundum) is used and one of the most susceptible materials to burning (marble) is cut.

The first type of coper is the turning-head coper, illustrated in Fig. 2. It is a machine very much like a regular planer, with a large, wide table. A wheel is placed on a swivelling head, enabling the operator to cut the marble in two directions at right angles to each other. This makes it possible

*President, the Royden Marble Machinery Co., 23d St. and Madison Ave., New York City.

FIG. 1. DRUM RUBBER FOR DRESSING MARBLE SLABS.
(The carborundum drum cannot be seen in the photograph, as it is below the bed of the machine. The roll visible is the rubber-covered pressure roll.)

to take the slab full size from the rubber and cope it to its exact rectangular dimensions. The arrises and edges left by this machine are not only equal but, in fragile marbles, superior to those left by the rubbing beds.

The second type of coping machine is the gang coper. In this machine the table is similar to that on the turning-head coper; the single head on the latter, however, is replaced by a slide containing twelve small heads. Each of these heads has a wheel which runs on a ball-bearing sleeve. A keyed shaft runs through the sleeves of all of the heads and operates them. It will be seen, consequently, that each wheel runs on its own bearing and that the danger of vibration, to which these wheels are very susceptible, is avoided. By a simple gear, these heads are automatically kept at the same distance apart. If one slab is cut into 6-in. strips and the next slab is to be cut into 8-in. strips, the operator, by turning a single wheel, resets all of the heads simultaneously. Where it is necessary to cut strips of different widths from the same slab, the heads can be disengaged from the spacing mechanism and placed in any position whatever. If the work requires a fewer number of wheels than are on the machine, the heads that are not to be used can be disengaged and pushed on the slide beyond the edge of the table. The strips are taken from the gang coper and placed upon the cut-off coper, where they are cut off to the requisite lengths.

A unique feature of these machines and, indeed, the device which has made coping by carborundum successful, is the use of dished wheels. Notwithstanding the fact that the wheels are only ⅝-in. thick, they are dished 1-32 in. on each side. Although at first sight it may appear that this is simply an adaptation of the regular wood saw, in reality it is not so. The sides of a carborundum wheel cut quite as well as its face, consequently it is not possible for the wheel to bind in the marble as a saw would bind in wood if its teeth were not flared. The advantage is due to the fact that the flare allows a slight lateral play of the wheel; consequently, even should the head vibrate considerably, it is not possible for the marble to nip the edge of the wheel.

Foreign inventors have been experimenting with carborundum for nine years, and have been unable to cope directly through the marble without such an enormous breakage of wheels as to make the process commercially unprofitable. The average life of a flat wheel is from one to two hours. The average life of a dished wheel under the same circumstances is about four days.

The slabs now go to the polishing machines. Marble is one of the most difficult and delicate materials to polish, and the successful production of a good polish requires a very accurate selection of the abrasive and the machine. Carborundum, which is unrivalled for sheer cuts, is a poor polisher. The aluminum oxides far surpass it.*

*See article by the author in Engineering News for June 6, 1907.

While it is possible to use a fine drum [illegible] The drum rubber, and so save much of the w[illegible] of polishing, it has been found more econ[illegible]dal to use a coarser drum which, though not producing so fine a surface, does very much more work.

Foreign marble workers have tried for [illegible]ny years to use disk rubbers, and so do away with a surfacing machine. They are, however, "chasing a rainbow," for it is a mechanical impossibility to secure an even abrasion with a disk machine. In England machines with as many as five disks, and in Belgium massive planetary disk polishers, have been introduced; but none of them successfully produce a true plane surface. Indeed, the work produced by them would not be accepted by the better class of American architects. Notwithstanding the greater artistic excellence of foreign designers, the workmanship on the more recent American marble interiors is the finest in the world.

The polishing operation is performed with hollow segmental disks, made of carefully selected bonds and grits of the aluminum oxides. The disks, though revolved mechanically, are swept over the surface by hand and, while automatic means for this purpose have been developed, they have not succeeded in producing the uniform abrasion which can be obtained manually.

The two-wheeled molder, shown in Fig. 3, differs essentially from the molding machines of foreign design, in its use of two wheels instead of one. It is due to this novelty that success has been attained. A coarse wheel has great cutting capacity, but leaves a poor finish. A fine wheel, on the other hand, leaves an excellent finish, but has very little cutting capacity and rapidly glazes if it takes anything more than a very light cut. The two-wheel process, by using a very coarse wheel to rough out and a very fine wheel (which takes only 1-16-in. cut) to finish, makes it possible to cut large molds easily and without undue expenditure of power. It does away with glazing, the great drawback to the single-wheel process. The wheels are cut to the desired shapes by means of a special steel dresser. Molds from this machine have fine, sharp edges, are not burnt in any degree, and have the same beautiful spine finish as left by the drum rubber.

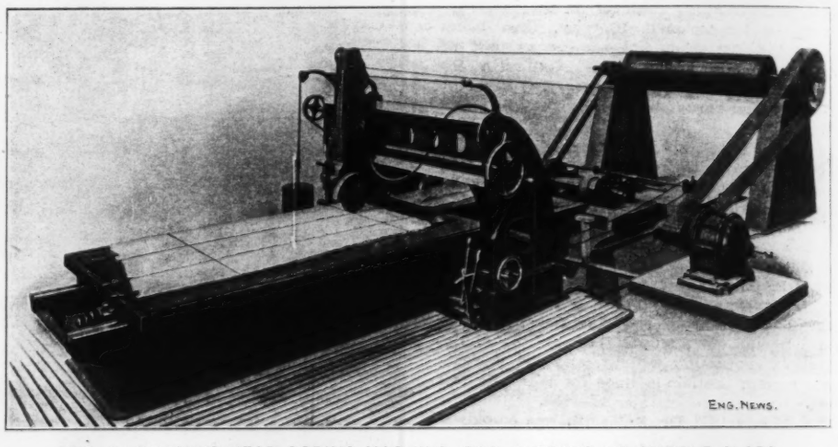


FIG. 2. TURNING-HEAD COPING MACHINE, FOR CUTTING UP MARBLE SLABS.
(The wheel may be turned so as to cut either longitudinally or transversely.)

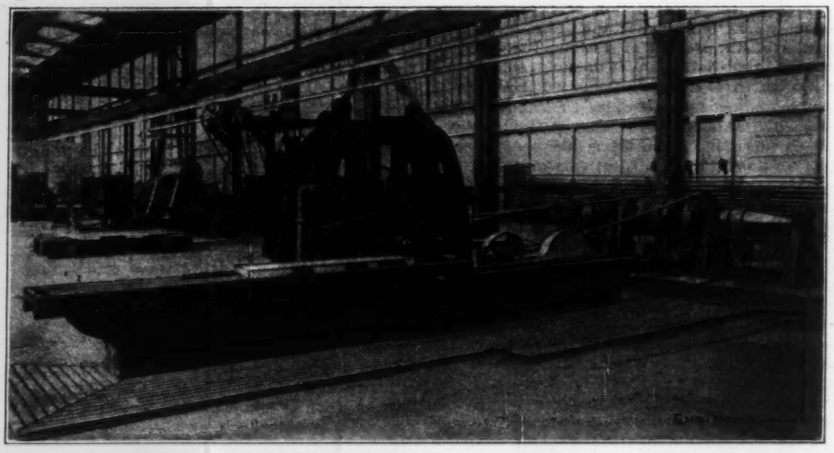


FIG. 3. TWO-WHEELED MOLDER FOR MARBLE WORKING.
(The wheel at the left is coarse, and takes a heavy cut; the one at the right is fine, and takes a 1/16-in. finishing cut.)

ELECTRIC OPERATION of the St. Clair tunnel of the Grand Trunk Ry. is expected to begin within 90 days. Westinghouse single-phase locomotives are to be used and the overhead conductor is now being put in place.

A FEED-WATER PURIFYING DOME for locomotive boilers is in experimental use on some freight locomotives of the 4-8-0 class on the Norfolk & Western Ry. The dome is set near the front end of the barrel, and is 17 ins. diameter, with a height of 18 ins. Within it are horizontal trays or baffle plates which collect a large proportion of the impurities in the water. Beneath the dome the boiler shell is pierced with 61 holes 1-in. diameter, and the water falls upon a deflector within the barrel. This serves to distribute the water in a finely-divided state over the surface of the hot water.

REINFORCED CONCRETE DIAPHRAGMS FOR EARTH DAMS.

By B. M. HALL, M. Am. Soc. C. E.*

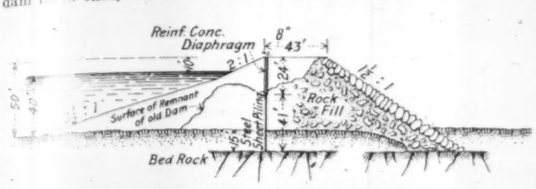
The Avalon Dam of the Carlsbad Project, U. S. Reclamation Service, Carlsbad, N. Mex., which was designed and constructed under the supervision of the writer, is an earth and loose rock dam, 50 ft. high and 1,050 ft. in length. This dam is, in fact, the reconstruction of an old dam which was destroyed by a great flood in October, 1904. The new dam has larger spillways, a heavier section and a core wall. The old dam had no core wall. Bed rock is from 12 to 19 ft. below the river bed or practically 65 ft. below the crest of the dam, and the core wall proper extends from bed rock to an elevation 24 ft. below the crest of the dam. This core wall is composed (longitudinally) partly of rubble concrete and partly of heavy interlocking-channel-bar steel sheet piling, the latter being driven to bed rock in such a manner as to form a core wall or diaphragm in that part of the old dam where it was thought best not to disturb the old fill by the deep trenching that would be required for constructing a concrete or masonry core wall. From the top of the steel piling and of the rubble concrete core wall just described, up to the crest of the dam, there is a vertical diaphragm of reinforced concrete, 12 ins. thick at its bottom, 8 ins. thick at its top, 24 ft. high and 1,050 ft. long. The reinforcing is composed of ½-in. square open hearth steel rods, spaced 12 ins. vertically and horizontally. Fig. 1 is a section showing the core or diaphragm of steel sheet piling with the reinforced concrete diaphragm resting on top of it. In designing the dam it was necessary to take into consideration the remaining parts of the old dam and utilize them to best advantage. This accounts for the use of sheet steel piling. Where rubble concrete, instead of steel sheet piling, was used the core wall was 7 ft. thick at the base, 3 ft. thick at the top, and had a maximum height of 45 ft., the same as the sheet piling.

It is a well-recognized fact that a durable core wall or diaphragm of some kind should be placed in every earth dam to prevent burrowing animals from making tunnels through the dam that will enlarge rapidly as soon as a stream of water begins running through, and to prevent definite water channels through the dam from any other cause. So far as the writer is informed, no advocate of core walls has ever claimed that even the heaviest walls in use are intended to add anything to the strength of the earth dam, or that the section of an earth dam could be safely reduced on account of having a masonry or concrete core wall in it. This being the case, it is evident that a diaphragm made of imperishable materials, and having a certain amount of flexibility, will fulfill all the requirements of the ordinary core wall and will have the additional advantage of being able to accommodate itself to slight inequalities of settlement in the dam. A partial recognition of this idea is shown by the fact that diaphragms of sheet metal and even of timber are used in some large earth dams.

There can be no doubt that a diaphragm of reinforced concrete will be more durable than either wood or sheet metal, provided it can be made with sufficient strength and flexibility. The writer has recently designed an earth dam for the proposed Carite Reservoir in the island of Porto Rico, in which it is planned to use a vertical concrete diaphragm 6 ins. thick, reinforced with ½-in. steel rods spaced 1 ft. apart, both vertically and horizontally. The dam will be 92 ft. high and the diaphragm is to extend from bedrock to the crest of the dam. The maximum section, as planned, is shown by Fig. 2. It is believed that this diaphragm will have sufficient flexibility to yield to the inequalities of pressure that may be caused by any lack of uniformity in the settlement of a well-constructed embankment, and that no serious break will occur in it. It is the intention to carry up the diaphragm along with the embankment, using inexpensive movable forms about 30 ins. in height, making a very wet mix for the concrete, and thoroughly puddling the earth on each side.

Any diaphragm or core wall should extend entirely to the crest of the dam, as a channel made by a burrowing animal above the ordinary water level would probably cause the destruction of the dam if the water should reach it during a flood.

*Consulting Engineer, 413 Temple Court, Atlanta, Ga.



1. SECTION OF AVALON DAM, CARLSBAD, N. M.
(In a part of the length of the dam rubble concrete was used in place of the steel sheet piling.)

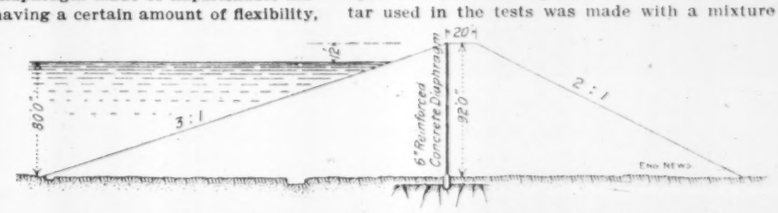


FIG. 2. PROPOSED EARTH DAM FOR CARITE RESERVOIR, PORTO RICO, WITH REINFORCED-CONCRETE DIAPHRAGM.

THE GRAND CANYON OF THE COLORADO in Arizona was made a national monument by executive proclamation on Jan. 23. The area included is approximately 825,000 acres and the boundary is set at about one mile back of the rim of the canyon.

TESTS TO DETERMINE THE EFFECT OF MICA ON THE STRENGTH OF CONCRETE.

By W. N. WILLIS.*

The accompanying curves give the results of a series of tests which have been recently made by the writer in the laboratory of the South & Western R. R. to determine the effect of mica upon the tensile strength of concrete. The mortar used in the tests was made with a mixture of 1 of cement to 3 of sand or sand and mica, by weight, the sand being either the pure Ottawa sand prescribed by the American Society of Civil Engineers specifications or Pittsburg standard crushed quartz. The various amounts of mica were added to the mixture in percentages, by weight, of the sand.

In many parts of the country, especially in sections of North and South Carolina, Tennessee, Virginia and Pennsylvania, the sand contains so much mica as to be worthless for concrete, the coarser sand containing from 3% to 4% and the finer sand, which at best is poor for concrete work, containing from 10% to 15%. The water in grading the sand, to a certain extent, separates the mica from the sand, leaving but a small per cent of mica in the coarser sand which is first deposited, but depositing the greater part with the fine sand.

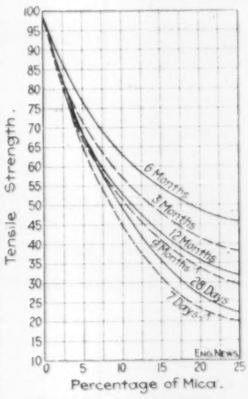


Fig. 3. Curves Showing Percentage of Decrease in Tensile Strength of Mortar Containing Varying Percentages of Mica.

Sand containing mica shows quite an increase in voids, as well as decrease in strength. The Ottawa sand used in these tests, loosely shaken, contains 37% voids, but increases gradually until the sand, which contains 20% mica, has

*Cement Tester, South & Western R. R., Johnson City, Tenn.

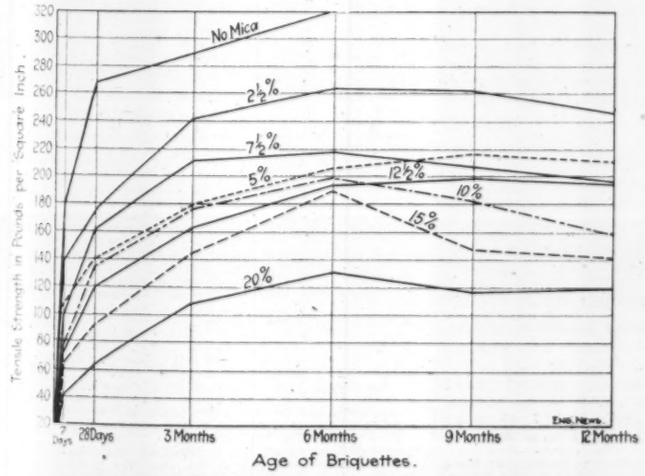


Fig. 1. Standard Ottawa Sand Mixture, 1:3 by Weight.

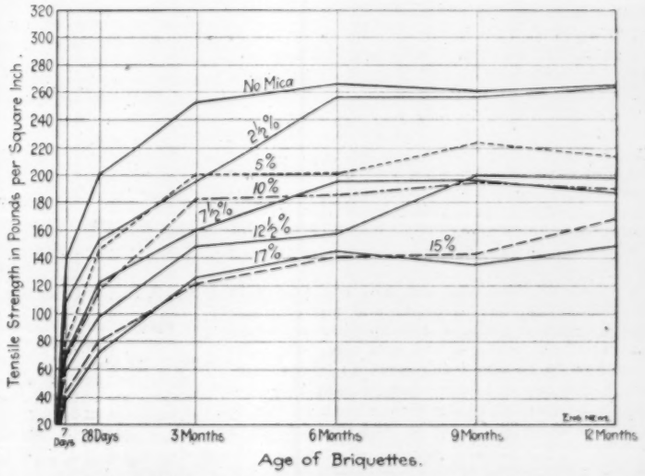


Fig. 2. Pittsburg Crushed Quartz Mixture, 1:3 by Weight.

FIGS. 1 AND 2. CURVES SHOWING EFFECT OF MICA IN SAND ON TENSILE STRENGTH OF CONCRETE.

67% of voids, and at the same time shows a decrease of 20% in weight. The increase in voids alone will decrease the value of the sand for concrete, while the surfaces of the mica are slick and do not make a good bond with the surrounding cement. As a general thing, the heavier a sand, the better it is fitted for concrete, grading being the same. Sand which contains 20% mica requires three times the amount of water for mixing that the same sand does without mica.

The mica used in these tests had a specific gravity ranging from 2.45 to 2.60, averaging 2.55, and was ground until all passed a No. 10 standard sieve, 29% a No. 20 sieve, 10% a No. 50 sieve and 4.5% a No. 100 sieve, making it as near as possible like the mica in the natural sand.

The tests made with Ottawa sand (Fig. 1) were made from four sets of tests, made in two parallel series about six months apart, and the averages taken, while the second series with crushed quartz (Fig. 2) were made from one set of tests, two and three briquettes only being broken at each period; not enough to equalize the faults from imperfect mixing, etc. The curves in Fig. 3 show graphically the decrease at each period of time.

A TANGENTIAL SUPPORT FOR TROLLEY WIRES.

In the London "Electrician" of Jan. 10 appears an article on a new and evidently very effective support for trolley wires, the invention of Mr. P. J. Pringle, Borough Electrical Engineer and Tramways Manager of the Burton-on-Trent Corporation.

The scheme on which the designs shown in Figs. 1 and 2 have been worked out lies in supporting the trolley wire by fittings so that the suspension wires are nearly tangential to the trolley in both horizontal and vertical planes. The ears for attaching the pull-off suspension

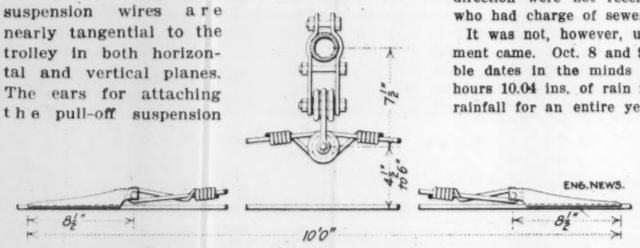


FIG. 1. DOUBLE TANGENTIAL TROLLEY SUPPORT.
(Devised by Mr. P. J. Pringle, M. Inst. E. E.)

for curves differs considerably from the straight-line ear as is seen by comparing Figs. 1 and 2. By proper designs of ears it is stated that the tangential stresses are carried in such a way that the trolley line is not distorted and curves of any radius can be equipped so that the ears do not pull over sideways or interfere with the trolley wheel. The photographs shown in the article mentioned and in a similar one in "Electrical Engineering" of Jan. 9, show that the suspension wire may simply pass over the insulator or may be tied as indicated in Fig. 1.

The spool shaped insulator used is of brown porcelain. From Fig. 1 it might be expected that the insulation would become impaired in wet weather or in dirty locations. However, Mr. Pringle states that he has had 50 pairs under a 500-volt test (length of test not stated) where they were exposed to the weather and to any dirt that might collect. In a very heavy rain the leakage was about 20 milliamperes and on cessation of the rain the leakage immediately dropped nearly to zero. The same type was under test for several months at 2,000 volts with satisfactory results.

It is stated that the sag and the angles in the trolley line, due to bends and pull-offs, are greatly reduced with a consequently smoother running of the trolley-wheel and pole. The cost is claimed to be about the same as in the ordinary methods. It is also claimed that this suspension is adapted, in the form shown in Fig. 1, to speeds up to 30 mi. per hr. and that for above that value a quadruple suspension is even better than the more expensive catenary construction. This statement, of course, remains to be demonstrated.

FRAUDULENT CLAIMS FOR DAMAGES FROM OVERFLOWING SEWERS, BROOKLYN BOROUGH, NEW YORK CITY.*

Devices for extracting money from the city treasury are various. A few are clumsy, many artistic and a few positively ingenious. Of all that have come under my notice during the two years that I have been Comptroller, no scheme for plundering the public promised a more alluring future than the sewer claim field.

When I took office on Jan. 1, 1906, I was rather staggered at many things. One of them was a pile of sewer claims—about 18,000 in number—which claimed damages amounting in the aggregate to about $10,000,000.

The story is simple but engaging. July 5, 1901, was an oppressively hot day. Late in the afternoon a violent thunderstorm burst over the city and within the period of a few minutes millions of tons of water were precipitated out of the clouds. It was an abnormal storm and the sewers, designed to meet normal conditions, were overtaxed.

Following this a few claims were filed against the city for those whose property had been damaged. The number was insignificant as compared with later figures, but it served to arouse the cupidity of certain members of the legal profession. Hundreds of claims were filed for floods that were wholly the result of brain storms.

The system built up rapidly. Bands of runners scoured districts that were known to be flooded. They went largely among foreigners unfamiliar with our laws, induced them to sign and swear to papers in blank, and then filled in the papers, inserting amounts ridiculously large. These were all duly attested by notaries who did not scruple about the formalities imposed by law.

Toward the end of 1901 the possibilities of the scheme began to dawn in the minds of a few members of the bar. Miles† was the pioneer and claims from his office were delivered in bundles, in amounts ranging from $50 to $2,500. Examiners of the department found many of the claims utterly ridiculous, but their efforts in this direction were not received with enthusiasm by those who had charge of sewer claims within.

It was not, however, until 1903 that the splendid moment came. Oct. 8 and 9, 1903, will always be memorable dates in the minds of the weatherwise, for in 3½ hours 10.04 ins. of rain fell upon the city. The normal rainfall for an entire year is 48 ins.

During 1904, 4,101 claims were filed; in 1905, 6,759. Many had been settled out of court, others had been taken before the courts and judgments secured against the city. When I took office on Jan. 1, 1906, almost one million dollars had been paid out to claimants through settlements or judgments and about 15,000 awaited adjustment.

Miles had a regular typewritten form of claim, with blank spaces for names, dates, etc., but when the tide was at its flood these spaces were filled in by typewriting. As men do not ordinarily carry typewriting machines about with them, and as the claimants were ignorant of the amounts, the proceedings smacked of the manufactory. Men swore that they were women, according to some of the affidavits.

One of the most barefaced attempts at robbery was the filing of a claim for damage to a building, the location proving to be an open lot. Another fellow included a horse in his bill. When questioned regarding the damage that a horse could sustain through getting its feet wet, the claimant declared that the horse had been drowned. A junk dealer declared under oath that several tons of scrap iron and brass had floated away, and

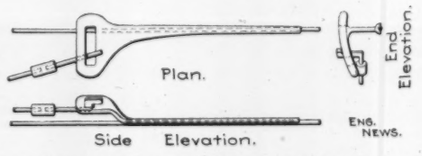


Fig. 2. Pull-Off Ear For Tangential Trolley Support.

an equally accomplished claimant averred that a ton of coal disappeared through the sewer pipe when the water ran off.

As to the results that have been accomplished through fighting these claims: In 1905, 6,739 claims, with an aggregate of $3,010,827, were filed. In 1907, 45 claims, with an aggregate amount of $55,235 were filed. In the past two years claims amounting to $1,4[illegible] have been absolutely withdrawn by claimants on t[illegible]ground in most cases that they were fraudulent and [illegible]e. To Jan. 1, 1906, the city had paid about one m[illegible]n dollars; since then it has paid $59,050, but a l[illegible] percentage of this amount was a legacy from th[illegible] days that came down through judgments obtained [illegible] and recently decided on appeal.

The system has been entirely reorganized [illegible] the handling of claims. In the old days two [illegible] three years often elapsed before a claim was exam[illegible]. During that time all evidences of damage, if t[illegible] had been any, were pretty well wiped out and the [illegible]y was placed at a disadvantage. We have divided [illegible] city into districts and every claim that is filed now [illegible] examined within 48 hours. Thus the examiner sees [illegible]he actual conditions and the city is not forced to r[illegible] upon the statements of claimants, which, unfortuna[illegible], are not always strictly truthful.

We have complete records of the rainfall fo[illegible] every day covering a period of several years back. These records were obtained from the U. S. Weather Bureau and have proved a distressing obstacle to claim[illegible]s who wish to recover for days that the records show to have been clear. Claimants are examined under oath as to all the particulars of their claims, and a record of the testimony kept. Thus, the sewer claim business has become insignificant.

Claims that are found to represent legitimate losses sustained under conditions that render the city liable are now paid promptly, and it is not necessary for any such claimant to pay a fee to any attorney. Any citizen in dealing directly with this office will receive just as prompt attention and as fair treatment as the most expensive lawyer can obtain for him.

A comparative statement, showing number of claims and amounts paid for, follows:

	Claims filed.	Amount claimed.	Amount paid.
1904	4,101	$2,419,561	$191,373
1905	6,759	3,010,829	300,252
1906	254	149,895	47,292
1907	45	35,235	11,768

These claims were re-examined between May 9, 1906, and Dec. 31, 1907, with the following results:

	No. of claims.	Amount.	Previous allowance.
*Allowance made	21	$11,829	$1,868
Repudiated as fraudulent	1,629	801,582	163,703
Disallowed	1,992	1,144,372	226,409
	3,642	$1,957,783	$391,979

*Present allowance, $1,652.24.

From Jan. 1, 1906, to Dec. 31, 1907, 2,805 claims amounting to $1,402,322, were repudiated and withdrawn by claimants and attorneys.

AN OFFICIAL STATEMENT ON THE RELATIONS BETWEEN THE NEW YORK, NEW HAVEN & HARTFORD R. R. AND THE WESTINGHOUSE ELECTRIC & MANUFACTURING CO.

The New York, New Haven & Hartford has officially denied the recent report that on Jan. 15, the Westinghouse Electric & Manufacturing Co. assumed the operation of the single-phase electric traction equipment on the New York Division between Stamford, Conn., and Woodlawn, N. Y. It is interesting to quote a few sentences from this official statement.

The electric service between Stamford and New York has not been discontinued, and at this time all Stamford local trains, as well as other local trains between Stamford and the Grand Central Station—with the sole exception of four trains to and from New Rochelle—are being handled by electricity. The entire matter has not been turned over to the Westinghouse company, nor is there any dispute of a financial or other nature between the two companies.

The entire operations, both of the power-house and the trains upon the line, are, as they always have been, in the hands of the New Haven. The Westinghouse company never has had any supervision over the operation of the railroad company's power-house, or trains. The connection with the operation of the electrical locomotives has been to provide experts for the instruction of engineers in the handling of the electrical machinery. Owing to some slight necessary changes in the locomotives, and to the fact that the nature of these changes demanded close technical electrical supervision, the Westinghouse company has charge of the small machine shop in which changes in and repairs to the motors are being made. It may be stated, however, that the locomotives furnished by the Westinghouse company have, in commercial service, exceeded the contract guarantee by 20% and under some conditions have been operated up to 100% in excess of the guarantees.

Certain changes were found necessary or desirable in the generators at the Cos Cob power station, in order to bring the station up to its full capacity. These are in progress, and should be completed before the close of the present month. The full capacity of the station, however, is much in excess of the operating requirements, and the installation has, at all times, been sufficient to provide for the electrical operation of the scheduled trains.

Both companies are working together in the closest harmony, all work is proceeding satisfactorily, and neither party has any doubt that the system will be handling both local and through service before the time provided by law that some power other than steam shall be used for operation through the tunnel in New York city.

*Condensed from a statement by Mr. Herman A. Metz, Comptroller of New York City, published in the Brooklyn "Standard-Union," New York City.

†Wm. O. Miles, a lawyer, convicted of sewer claim frauds in the courts.—Ed.

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Six years ago, in our issue of Feb. 20, 1902, we discussed the design of "diaphragm dams," and pointed out the advantages of substituting a thin water-tight diaphragm in the center of an earth dam in place of the ordinary puddle core or masonry core wall. Since then dams with steel plate diaphragms have been quite extensively used in the West, and in the present issue we describe a dam with a reinforced-concrete diaphragm built on one of the Reclamation Service projects. For cutting off flow beneath a dam, it is becoming increasingly common to use a diaphragm of sheet-piling. The introduction of steel sheet-piling enables such sub-surface diaphragms to be made with a certainty not possible with wooden piling. At the great Gatun Dam, as pointed out by Col. Goethals in our issue of Jan. 30, steel sheet-piling is to be put down to check possible seepage through the old river channels, although the base of the dam will be also made secure by a deep trench filled with puddle.

At a recent meeting of the American Institute of Electrical Engineers some of the most prominent men in that profession wrestled with the reformation of technical schools. By a survey of the epitome of the discussions at this meeting, printed elsewhere in this issue, it would seem that all discussions on this subject have reached a point where little new matter can be injected into them until definite action is taken by the educators themselves.

The educators and engineers alike have agreed that a change is necessary, and that this should be toward a substantial reduction of the number of studies taught and a corresponding simplification of courses. It is held that the fundamental principles on which electrical engineering rests should be reiterated, again and again, from various viewpoints and that commercial practice should be given little room in the curricula. There is a demand for more attention to those "liberal arts and sciences" which it is hoped will broaden the graduate's conception of his profession and its relation to men and things.

This is the statement of belief in what should be done. What is most needed now is some good technical school which has the courage of convictions; which will throw aside convention and teach engineering as the professors believe it should be taught. The peculiar spectacle now exists of instructors asking for a new order of studies, but teaching the old; demanding more serviceable curricula but clinging to the despised ones as drowning men cling to bits of wreckage. If the old courses of study are antiquated, they must eventually pass away and the instructor who has not the faith and courage to shape his course toward the new must disappear with the old.

Before further discussions on this subject can be profitable the truth of the assertions on record must be demonstrated. The present condition of technical education is worrying the electrical branch of the profession more apparently than any others; but the criticisms which have been made are applicable to all branches of engineering education.

The barge towing system, in extensive use on the Great Lakes and on the Atlantic coast for the transport of ore, coal and other bulk freights, involves heavy risk in case of storm. The severe storm of Jan. 23 caused the loss of several barges on the Atlantic coast; and the storms which are common near the close of navigation on the Great Lakes take heaviest toll of human life among the barge crews. These risks are reflected, of course, in higher rates for marine insurance and increase by so much the cost of carriage. In addition the floating derelict barges constitute a danger to other vessels, and it is a question worth consideration whether the use of barges should not be prohibited, or at least placed under restrictions in the interest of public safety.

The punishment of "common carriers" for joining together in a "traffic association" to maintain rates, which is made possible by the Sherman anti-trust law, is by no means so unprecedented as is generally supposed. It is a well known principle of common law, established centuries ago, that agreements in restraint of trade or to limit and restrict competition are against public policy and will not be enforced by the courts.

At a very early era in transportation development in this country, however, the courts went further than this and actually punished those who attempted to get rid of competition. An incident of this sort was reported some months ago in the Pittsburg "Gazette-Times." In 1842 the chief avenue of transport from eastern Pennsylvania across the mountains to Pittsburg was the Pennsylvania Canal, built and owned by the State. A number of the principal owners of boats on the canal formed an organization and agreed under oath to keep their rates uniform and not compete with each other. The Supervisor of the canal learned of the organization, secured testimony as to its existence and acts and had nine of the men belonging to it arrested and put on trial for conspiracy. All of them were convicted and sentenced to a fine of $100 to $150 each and two months' imprisonment in the common jail. After serving four days of their sentence, however, they were pardoned by the governor, but the fines against them were not remitted, which would indicate that the pardon was not intended to nullify the action of the court. That the offenders themselves recognized the illegality of their acts is also indicated by the fact that they promptly dissolved their organization as soon as they were released from jail.

We noted in our issue of Jan. 16 the forty years' service of Mr. Thos. Murray as City Engineer of Davenport, Iowa, and a fifty-year term of a city surveyor at Salisbury, England. Both these records are surpassed by Mr. David Earl Whitford of Syracuse, N. Y., who has been in the service of the Canal Department of the State of New York since March, 1852, with the exception of about four years during that period. A dinner was recently given to Mr. Whitford by his friends in the engineering profession and some account of it is given in our Personal column. These instances are worthy of note in view of the statement frequently made that engineers in public employment suffer frequent displacement for political reasons.

A correspondent from Minnesota writes us that a railway company is being organized there to be known as the "Thief River Falls & Roosevelt Air Line." We understand no reflection on the high financiers of Wall St. is intended; but it will probably be safer not to seek a market for the bonds there.

THE RESPONSIBILITY FOR SAFE CONSTRUCTION IN REINFORCED CONCRETE.

Every little while appeals come to us to do something, in the interests of public safety, toward stopping the construction of reinforced-concrete structures by men who are ignorant, incompetent or worse. These appeals come most largely from those engaged in reinforced-concrete construction who realize the great harm that is done the industry by every building of this type which collapses during construction.

Perhaps it will tend to the desirable end above set forth if we quote from the charge of a judge in Philadelphia before whom on Jan. 30 Albert S. Reavis, President of the Reavis Construction Co., and Chas. B. Miller, Superintendent, were tried on the charge of manslaughter because of the death of two workmen in the collapse of a reinforced-concrete building which the company was erecting. (Eng. News, July 18, 1907, p. 69).

The trial judge is quoted as follows in the Philadelphia "Ledger":

> The question is whether the death of these men was due to the gross carelessness or the utter incompetency of the defendants. In either case they were culpable. Houses are not built to fall down immediately. Were these men too ignorant to take up their business of reinforced concrete construction? If they were, it is a case of malpractice. Nobody has a right to hold himself out as qualified to do a thing unless he is able to do it.
>
> If these men didn't know enough to be in the concrete business, it was an impudent assumption on their part to risk the lives of men in the erection of this building. Or if it was stinginess which led them to remove the supports of the concrete too soon so that they could use these supports at another place and thereby save some money, they were equally culpable. If the removal of the supports was a mistake of judgment to such a degree as showed that they ought not to be in the business, they are just as guilty. Nobody out of ignorance has the right to risk the lives of other men.

It is worth noting that the attorney for the defense, according to the "Ledger" based his plea on the ground "that reinforced-concrete construction was in its infancy, and the defendants exercised as good judgment as possible in the existing state of knowledge of such construction."

If the reinforced concrete business is an infant, it is a pretty lusty one. In our opinion it has not only cut its eye teeth but grown to man's estate. There are plenty of engineers and contractors to-day who know how to build and build safely in reinforced concrete and anyone who fails to build safely through ignorance ought not to escape just punishment for the results of his blunders.

THE DROP IN RAILWAY TRAFFIC AND EMPTY CAR MOVEMENT.

Never was there such a sudden change in the volume of railway traffic as has taken place in the past three months. On Oct. 30 the statistics collected by the American Railway Association's Committee on Car Efficiency showed that the railways of the United States and Canada were short 90,000 cars of the number ordered by shippers. By Nov. 13 the total of shortages had been reduced to 57,000 cars and a total of 12,200 surpluses was reported. On Nov. 27, the shortages fell to 18,000 and the surpluses had grown to 40,450. By Dec. 24 the surplus cars were 119,000 and the shortages, confined chiefly to Canada and New England, were only 4,500.

In the next fortnight the surplus cars increased astonishingly, and on Jan. 8 the reports showed 341,683 idle cars on 160 roads. On Jan. 22 the number remained substantially the same, the total surpluses reported by 153 roads being 339,053 cars.

While the business depression is, of course,

the chief factor in the falling off of traffic, it must also be remembered that the close of navigation on the Lakes causes a large reduction in traffic on the lines to and from the Lake ports, the movement of grain is far less than during the fall months and the mild winter has made the coal traffic much lighter than the normal.

As our readers are doubtless aware, it is years since the railways as a whole have had any large surplus of cars. The problem of the traffic departments has long been how to find cars to meet the demands of shippers. Only a year ago the legislatures were teeming with bills to punish the railways for failure to promptly provide cars; but to-day some of the railway companies are actually building sidetracks on which to store their idle freight cars.

Last July a new per diem rate of 50 cts. per car went into effect, under which a railway has to pay 50 cts. per day for each day it retains a foreign car on its lines. This rate was an increase over the old per diem rate and was intended to give an increased incentive to a railway to return foreign cars to the roads owning them instead of retaining them to use in its own traffic. So long as business continued brisk, however, foreign cars continued to find the route homeward very long and roundabout, but when the sudden drop in traffic occurred, railway companies everywhere awoke to the fact that there was no object in holding foreign cars idle and paying 50 cts. a day rental on them. For the past few weeks, therefore, there has been an unprecedented movement of empty cars homeward and the waste of this needless haulage has become so evident that a special meeting of the American Railway Association has been called to act upon a change in the per diem rules to take effect on March 1, next.

Exactly this condition was foreseen by the Car Efficiency Committee of the Association. In its report of Oct. 28 last, we find the following:

> Too much should not be expected from the per diem rate. Of course, a proper adjustment of the rate is a factor in car efficiency, but it is by no means the only factor. In the effort to secure a rate which will be compensatory to the car owner when he has use for all his cars there is danger of fixing the rate at a point where it will become burdensome when cars are easy, or greatly increase the empty haul, which would be wasteful.

It will be evident that when a car owner has to build additional sidetracks to hold idle cars, he prefers not to have them sent home and it is more economical for all concerned to hold them wherever they may happen to be until an opportunity offers to load them homeward than it is to send them all the way home empty, as is being done at the present time.

THE COST OF STORING THE LAST DROP OF WATER FROM THE CROTON DRAINAGE AREA, NEW YORK.

The new Catskill water supply for New York City was planned and is being carried out to avert the danger of a water famine. During the past year, however, the rainfall on the Croton drainage area, from which New York draws its present supply, has been phenomenally heavy, and the sight of water running to waste over the great Croton dam has started off one of the daily newspapers on a crusade to prove that the Catskill water supply is a huge mistake and that the city could draw all it requires from the Croton, were proper storage provided.

It supports this proposition by some computations in which it obtains the total annual yield of the Croton basin by the simple process of multiplying its area in square miles by the total average rainfall, making no deduction whatever for evaporation even. Hydraulic engineers would be happy, indeed, were they able to collect water under such ideal conditions; but certain immutable laws of nature have always stood in the way. Besides evaporation, floods must be considered, and every engineer knows that it is utterly impracticable to utilize the total flow of a stream which is used for public water supply.

As the average citizen and many city officials are ignorant of these matters, it sometimes becomes necessary for the water supply engineer to explain why storing the last drop of runoff from a drainage area would be far more costly than going much farther afield for an additional supply. Such an explanation, or demonstration, based on the unusually complete rainfall and runoff records of the Croton drainage area, New York City, for the past 40 years, appears elsewhere in this issue. The author of the article, Mr. Alfred D. Flinn, M. Am. Soc. C. E., also shows the cost of storing the surplus waters in the dryest and again the wettest 18-year period of the 40 years, 1869 to 1907, inclusive. For the detailed figures, our readers are referred to Mr. Flinn's interesting article. We need only note here that to store all the water that will go to waste after the Croton drainage area has been developed to its economic limit would add but 14% to the otherwise available supply and would cost nearly as much, for storage alone, as the total estimated cost of the Catskill supply, of half a billion gallons a day, which is now being provided.

This is not all. For many years out of a period like the last 40 years, some of the reservoirs would be empty and their bottoms would become covered with vegetation which would have to be removed at no little cost or else might give rise to bad tastes and odors in the water when the reservoirs were again filled.

Obviously, storing every drop of water that reaches the Croton, or any other stream under like circumstances, would be absurd. In other words, as we have already intimated, and as every water-supply engineer knows, there is an economic limit to the amount of storage that can be provided in a given drainage area. The real governing condition, too, is not the maximum rainfall, nor yet the average for the period covered by available records, but the *minimum* rainfall and runoff over the dryest series of years in the whole period. For of what use is it to build reservoirs, and pay capital and maintenance charges thereon, only to have the reservoirs empty in the very years when the natural flow of the stream is most deficient?

With this in mind, Mr. Flinn shows that in the dryest 18 of the 40 years between the beginning of 1869 and the close of 1907, to store all the water that would not be held by reservoirs now actually or nearly completed would add less than 3% to the supply and that at a cost of $12,500,000. The foregoing remarks relate to only a few of the many interesting points brought out by Mr. Flinn's article, which we are sure will be appreciated by every engineer interested in public water supplies.

THE NEW YORK BARGE CANAL VS. THE DEEP WATERWAY.

It has been frequently pointed out in these columns that the New York barge canal was planned and has been furthered, from the beginning to the present time in the interest of Buffalo, or more definitely of the interests which control the business of freight transferring there. It is treason in Buffalo to talk of a deep waterway which would permit lake vessels to sail past her doors and go down to the cities on the Atlantic seaboard to discharge their cargo. Every effort of the Board of Advisory Engineers to secure greater depth for the barge canal has been fought by Buffalo, which lives in fearsome dread that by some means or other a way will be found for freight to go by her without paying toll.

Naturally, therefore, the recent report of Superintendent of Public Works Stevens, advocating the Federal deep waterway and showing that the barge canal on which New York is expending a hundred million dollars is already obsolete when hardly begun, has excited Buffalonians as a red rag excites a bull.

Since it appeared, the Buffalo newspapers have seethed with indignant utterances. Supt. Stevens' report has been characterized as "the veriest rot," and he himself has been held up to public condemnation as a malefactor of deepest dye. It is not worth while to notice such campaign literature, except for the striking evidence it affords that it is local interests at Buffalo and not the larger transportation needs of the state that the barge canal is designed to serve.

There are, however, some gross misst[illegible] in this Buffalo literature which do mer[illegible] tion. Over and over again it is asser[illegible] that the Federal deep waterway has been [illegible] gated by experts and found impractica[illegible] The following quotation will serve as an ex[illegible]:

> The idea of building a ship canal with F[illegible] was long discarded. For years that idea was e[illegible] and in 1899 or 1900 it was thoroughly inves[illegible] a committee of men capable and competent to [illegible] as to its merits or demerits. They reported it to[illegible] lutely impracticable.

Now, unfortunately for the reputa[illegible] for truthfulness of the author of the abov[illegible] tatement, the Federal deep waterway ha[illegible] been shown to be entirely practicable, and tw[illegible] ifferent committees of capable and compete[illegible] engineers have so reported. Their opinions an[illegible] tatements did not rest on mere guesswork [illegible] ither, but on the most thorough surveys and in[illegible] stigations in which nearly a million dollars [illegible] was expended. The Buffalo supporters of the barge canal are either ignorant of the facts gi[illegible]n in the Federal Deep Waterway Commission report of 1900 or else they are deliberately making misstatements concerning it. They have made wholesale accusations of ignorance or malice against Supt. Stevens, which they might well take home to themselves.

Compared with Buffalo, New York City's interest in the barge canal work is lukewarm. However, the interests which control the freight transfer business were also aroused by Supt. Stevens' report. They have also in the past lent their influence against the Federal deep waterway for fear that lake vessels might sail down the Hudson and out of the bay and discharge cargo at the wharves of Boston or Providence or Philadelphia or Baltimore. The "Canal Association" of New York City made public on Jan. 31, an attempted answer to Supt. Stevens, in which it stated among other things that:

> A 21-ft. canal from Lake Ontario to the Hudson River would necessitate a corresponding depth in the channel of the Hudson River, which could not be obtained and maintained, either by State or National Government, except at enormous cost.

We have been interested to compare this statement with the actual estimates for this section made by the Deep Waterway Commission of 1900, after exhaustive surveys. On page 82 of the report of that Commission is given the estimated cost of a 21-ft. deep waterway in the Hudson from the mouth of Norman's Kill (just below Albany, where the canal section of the deep waterway begins) to New York City; and the "enormous cost" to which the Canal Association refers, is the sum of $4,160,000!

This is a fair sample of the misrepresentation which is being used to bolster up the barge canal project, and prevent the people of New York and their representatives at Albany from learning the truth concerning this $100,000,000 bunco game.

That these barge canal boomers will succeed for the present seems entirely probable. The private interests which have millions of dollars at stake can finance enough publicity to mislead most of the people and can probably secure the backing of enough commercial organizations throughout the state to squelch any further talk of a Federal deep waterway at Albany. No one on the other side has any financial interest in arousing opposition, enlightening the people as to the extent which they are being deceived and defrauded, and securing an expression of public sentiment favorable to the Federal deep waterway.

We doubt, however, if the success of the barge canal boomers will be permanent. The sentiment for waterway improvement is growing throughout the nation. The other Atlantic coast states in convention at Philadelphia last fall, developed great enthusiasm for an inland waterway parallel with the coast. They will eventually awake to the fact that a deep waterway from the seaboard to the lakes will be many times as valuable to them. So, too, the whole northwestern and southwestern region tributary to the Great Lakes will eventually demand a waterway to its markets on the seaboard on which freight will not have to pay tolls for transfer at any city on its way.

LETTERS TO THE EDITOR.

A Cheap, Self-Adjusting Measuring Weir for Irrigation or Other Use.

Sir: Referring to the need for a cheap method of measuring water, set forth by Mr. Means in your issue of Nov. 21, 1907, the following article, which I copy from the "Irrigation Age" of March, 1903, will, I think, suggest a way out of the difficulty. The patent spoken of in the article has not been exploited by the inventor, owing to the lack of funds.

Very truly yours,
E. R. Weeks.

Cheyenne, Wyo., Dec. 2, 1907.

[The article mentioned by our correspondent stated that a new weir had been invented by Mr. C. C. Carlisle, who was at that time Assistant State Engineer of Wyoming. Inquiry on our part has shown that Mr. Carlisle, who is now City Engineer of Cheyenne, Wyo., secured U. S. Patent No. 706,526 (Aug. 12, 1902), in which he claimed two improvements on the Marseilles module, used but abandoned on the Marseilles Canal, France, and in Australia and the United States. The improvements claimed were (1) a mercurial seal for the adjustable float-supported submerged cylindrical weir, and (2) the admission of air just beneath the top of submerged weir, to prevent siphonic action.—Ed.]

A Table of Short-Radius Curves.

Sir: The accompanying table of short-radius curves may not be at all a new thing, yet I have never seen a similar one except those made by men on my parties. For trolley location, pipe line work, curved street lines and in many other ways I have found it useful.

The chord length is so short that the line is practically an arc. The *P. C.* and *P. T.* are located from the *P. I.* by the relation

$$\text{Tangent} = R \tan \tfrac{1}{2} I.$$

For length of curve, divide the total angle by the central angle of the chord, and multiply by the length of chord. Stakes are usually set on the even stations and plusses equal to chord length. Use "Deflection for 1 Ft." in figuring odd feet and tenths at beginning and end of curve.

Radius, ft.	Chord, ft.	Central angle.	Deflection angle.	Deflection for 1 ft.
35	10	16°26′	8°13′	0°49.3′
45	10	12°46′	6°23′	0°38.3′
50	15	17°16′	8°38′	0°34.5′
60	15	14°22′	7°11′	0°28.8′
75	15	11°30′	5°45′	0°23.0′
100	20	11°30′	5°45′	0°17.3′
120	20	9°34′	4°47′	0°14.3′
150	20	7°39′	3°49′	0°11.5′
190	25	7°32′	3°46′	0°09.15′
200	25	7°10′	3°35′	0°08.6′
225	25	6°25′	3°12′	0°07.7′
240	25	5°58′	2°59′	0°07.2′
250	25	5°44′	2°52′	0°06.9′
275	25	5°12′	2°36′	0°06.2′
288	50	9°58′	4°59′	0°06.0′
300	50	9°32′	4°46′	0°05.7′
350	50	8°12′	4°06′	0°04.9′
376	50	7°40′	3°50′	0°04.6′
400	50	7°10′	3°35′	0°04.3′
410	50	7°00′	3°30′	0°04.2′

E. W. Hess.

Clearfield, Pa., Jan. 3, 1908.

The American Institute of Electrical Engineers on Forest Preservation.

Sir: I enclose herewith a copy of the resolutions on forest preservation adopted by the Board of Directors of the Am. Inst. E. E., Jan. 10, 1908. It is my personal opinion and that of other members of this committee that these resolutions apply with especial emphasis to the Appalachian watersheds. I know from personal observation in New Hampshire during the last fifteen years that national forests in these regions are urgently required for timber supply as well as for the protection of water powers.

I believe strongly that the bill H. R. 10457, providing for national forests in the Appalachians deserves the active support of the engineering press and of engineers as individuals.

WHEREAS, The American Institute of Electrical Engineers recognizes that water powers are of great and rapidly increasing importance to the community at large, and particularly to the engineering interests of the country; and

WHEREAS, The value of water powers is determined in great measure by regularity of flow of streams, which regularity is seriously impaired by the removal of forest cover at the headwaters with the resulting diminution in the natural storage capacity of the watersheds, this impairment frequently being permanent because of the impossibility of reforestation, owing to the destruction of essential elements of the soil by fire and its loss by erosion; therefore

BE IT RESOLVED, That it is the opinion of the American Institute of Electrical Engineers that the attention of the National and State Governments should be called to the importance of taking such immediate action as may be necessary to protect the headwaters of important streams from deforestation, and to secure through the introduction of scientific forestry and the elimination of forest fires the perpetuation of a timber supply; and further

BE IT RESOLVED, That the Committee on Forest Preservation be instructed to communicate these resolutions to all Members of Congress and to the Governors of all the States.

Very truly yours,
Charles H. Porter.

Mass. Inst. of Tech., Boston, Jan. 28, 1908.

The Cameron Septic Tank Decision Not Accepted as Final by Saratoga Springs.

Sir: In Cameron vs. the Village of Saratoga Springs, a motion was made by Mr. C. L. Sturtevant, of Washington, D. C., in behalf of the defendant, on Jan. 22, 1908, before Judge Ray for an order suspending the issuing of an injunction restraining the use of the septic tank process, described in the Cameron patent. [See Eng. News, Jan. 16, 1908.—Ed.] The motion was opposed by Mr. Gifford, attorney for the Cameron Septic Tank Co., and the matter is now pending. The defendant proposes to move before the Circuit Court of Appeals for a rehearing, and if that is denied, apply to the U. S. Supreme Court for a writ of certiorari to review the action of the Circuit Court of Appeals. It is conceded by the complainant that if the defendant cleans out its tanks as often as once in three months it will not infringe.

It seems strange that a municipal corporation cannot run its sewage into a tank, which the court holds is not patentable, and there allow it to remain, so long as the public health is not affected.

E. F. Musson, M. Am. Soc. C. E.

Norwich, N. Y., Feb. 1, 1908.

Suggestion for a Working Agreement Between Engineers and the Cameron Septic Tank Co. as to the Use of the Septic Process Patents.

Sir: In view of the decision of the U. S. Circuit Court of Appeals in the matter of the Cameron septic tank process, I beg to suggest to the profession and others interested that a committee of engineers be delegated to take the matter up with the Cameron Septic Tank Co., with a view of coming to some amicable adjustment, whereby an engineer or engineering firm may acquire the right to deliver the right of use of this process to its clients, so as to place the profession on an equal basis in the matter of cost of the aforesaid rights. It could be placed upon a per capita basis insomuch that the cubical contents of a septic tank varies considerably, as engineers differ much in respect to the sizes thereof.

The Cameron Co. would surely act in a highly professional spirit in the matter, and would be bound to compete on similar terms with principals.

The adjustment of claims on plants already installed could be made upon some acceptable basis.

I submit the foregoing with the idea of eliminating future litigation and of clearing up the situation, in order that the septic process may be advocated by engineers and included in their designs on unquestionable lines, as best conserving their clients' interests as well as their own.

Augustus C. Hepp,
Chief Engineer, American Dittler Sewage Disposal Co.
1123 Broadway, New York City, Feb. 1, 1908.

[This proposal should be read in connection with the preceding letter, in which it is stated that an effort is being made to secure either a rehearing of the septic tank case or else an appeal to the U. S. Supreme Court. In our issue of Jan. 16, 1908, we assumed that the decision was final, but it seems that plucky little Saratoga Springs is not yet ready to admit defeat.—Ed.]

Additional Information on a Short Method of Re-Computing Sewer Discharges.

Sir: I was very much interested in Mr. Coleman's paper in your issue of Nov. 21, 1907, on "A Short Method of Re-Computing Sewer Discharges for a Changed value of n in Kutter's Formula." It seems to me that the methods suggested therein might frequently be of much use to engineers having similar problems, but I have been unable to understand the diagram accompanying the paper, or to find out how to make use of it. Mr. Coleman carefully explains how to compute the ratios between the discharges with different values of n, and then presents a diagram in which the x lines represent the values of n and the y lines represent —what? It is nowhere stated what the figures 1.0, 1.1, 1.2, etc., represent, or how they are to be used. I should have inferred from the earlier text that they might be multipliers to apply to the tabulated values of the discharge where n equals .012, but in this case diagonal lines should intersect the line for $n = .012$ at a value of unity, and this is not the case.

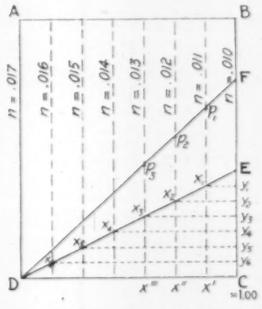
Very truly yours,
Charles W. Sherman.

14 Beacon St., Boston, Mass., Dec. 2, 1907.

[On submitting Mr. Sherman's letter to Mr. Coleman the latter replied as follows:—Ed.]

Sir: I am much obliged to Mr. Sherman for his kind remarks and I think the following explanation will make everything quite clear. I certainly did not go into this portion of the work so fully as I might have done had I put myself in the place of one taking up the subject for the first time.

The paragraph immediately referring to this portion of the work commences, "It then became necessary to obtain an easy method," etc. I will now proceed to elaborate the rest of the paragraph.



Draw a base line DC of any convenient length. Set up the verticals DA, CB, for the present of indefinite length. Calculate from first principles the discharges of a 10-ft. diameter sewer, at any chosen gradient (the flatter the better), for each value of n from $n = .010$ to $n = .017$. Find the ratios which each of these discharges bears to the discharge when $n = .017$. These will be 1.724, 1.564, 1,430, 1.317, 1.221, 1.137, 1.064, 1.000; i. e., taking the discharge for $n = .017$ as unity we find the values for the other values of n. Call the point C unity, and plot up CB to any convenient scale the values given above. Let E be the point corresponding to $n = .010$. Join ED. Through the points y_6 y_5, etc., draw horizontals to cut the 10-ft. diameter line ED in x_6 x_5 x_4, etc. Draw verticals through x_1 x_2 x_3 x_4 x_5 x_6. Take any other sewer, say one 7 ft. in diameter, and calculate the ratio between discharge when $n = .017$ and $n = .010$. This will be about 1.7584. Plot this value up CB. Let it be CF. Join FD. Then just as the heights x_1 x', etc., give the multipliers to convert from $n = .017$ to any other discharge for a different value of n when the sewer is 10 ft. in diameter, so the values $p_1 x'$, $p_2 x''$, etc., will give the multipliers for the 7-ft. sewer, the discharge when $n = .017$ being taken as the base value. For any intermediate values, say for instance to convert the discharge when $n = .013$ to when $n = .011$; find the discharge when $n = .013$ and multiply it by the ratio * $p_1 x' : p_2 x'''$, and so on.

I find on rereading the article that the formula beginning on line 28, p. 552, col. 1, should read:

$$\frac{V_1}{V_2} = \frac{\left\{\frac{l}{n_1} + a + \frac{m}{s}\right\}\left\{\sqrt{R} + \left(a + \frac{m}{s}\right)n_2\right\}}{\left\{\sqrt{R} + \left(a + \frac{m}{s}\right)n_1\right\}\left\{\frac{l}{n_2} + a + \frac{m}{s}\right\}}$$

not

$$\frac{V_1}{V_2} = \frac{\left\{\frac{l}{n_1} + a + \frac{m}{s}\right\}\left\{\sqrt{R} + a + \frac{m}{s}\right\}n_2}{\left\{\sqrt{R} + a + \frac{m}{s}\right\}\left\{\frac{l}{n_2} + a + \frac{m}{s}\right\}n_1}$$

Yours faithfully,
G. S. Coleman.

Town Hall, Manchester, England, Dec. 15, 1907.

The Strength of Columns as Shown by Buchanan's Tests.

Sir: I have read with much interest the report of tests of steel compression members in your issue of Dec. 26, 1907, and also the editorial article on p. 702 of the same issue, commenting on the same.

The first peculiarity of the results, noticed by the writer, was (from the photographs) that the chord sections nearly all failed on the latticed side, and that the deflection was upward in all cases. The cause of this was immediately apparent when it was noted that the chords were tested on their sides with the pins vertical (see footnote on p. 686) and that the pins were placed away from the neutral axis, to compensate for the bending due to the weight of the member when in place in

*The line DC is taken as unity for the verticals instead of taking it as zero, as the values for the verticals never fall below unity and so the part of the vertical between zero and unity is useless and need not be plotted. The values for $p_1 x'$, $p_2 x''$, therefore must be their values, taking DC as the unit line.

the bridge. Thus, in the tests, these chord sections were all eccentrically loaded columns.

For this reason, it appears to the writer that the conclusions drawn in the article on p. 702, that the columns failed at from 80% to 90% of the compressive strength of the material, are erroneous, because 13 out of the 14 short columns tested were these eccentrically loaded chord sections.

The maximum fiber stress in an eccentrically loaded column is easily figured when the deflection of the member is neglected.

Let $S_{max.}$ = the max. fiber stress.
P = total load.
A = area of cross-section.
M = bending moment.
v = distance of extreme fiber from neutral axis.
I = moment of inertia.
r = radius of gyration.
$s_c = \frac{P}{A}$ = average unit stress.
e = eccentricity.

Then $S_{max.} = \frac{P}{A} + \frac{Mv}{I}$; $M = Pe$; and $I = Ar^2$.

or $S_{max.} = \frac{P}{A} + \frac{Pev}{Ar^2}$ or $\frac{P}{A} = s_c = \frac{S_{max.}}{1 + \frac{ev}{r^2}}$.

Using for the value of $S_{max.}$ in this equation the elastic limit of the material, 30,700 lbs. per sq. in. for iron and 39,900 lbs. per sq. in. for steel, we get the results given in the table below.

The values of r, e, and v, when obtainable, are taken from the report of the tests as given in Mr. Buchanan's article, otherwise they were calculated from the drawings. The stress for columns No. 18 and No. 19 could not be calculated as the location of the pins is not given on the drawings.

Test No.	r	v	e	s	Calculated. $\frac{P}{A}$	Actual unit load.		Per cent. of calculated.	Kind of failure.
5	5.55	9.18	0.58	30,700	26,180	23,290	I	89.0	End de-
6	4.88	7.99	0.49	"	26,370	28,220	I	107.0	tail.
7	5.69	8.74	0.49	"	27,110	29,210	I	107.7	"
8	5.69	8.74	0.49	"	27,110	29,990	I	110.3	"
10	6.34	10.27	0.62	"	26,510	26,200	I	98.8	"
11	6.33	10.29	0.64	"	26,370	26,230	I	99.4	"
12	5.65	9.09	0.68	"	25,710	30,770	I	119.7	"
13	5.80	8.44	0.39	"	27,900	28,200	I	100.9	Body.
14	4.71	7.95	0.95	"	22,900	26,260	I	114.7	"
15	6.51	9.86	0.38	"	28,200	30,300	I	107.5	"
16	6.44	10.36	0.74	39,900	33,670	30,640	S	90.9	"
Average for iron....					26,442	27,858		105.3	
Average for body failures.....								103.5	

The percentages given above show that the actual fiber stress at crippling varied from 89% to 119.7% of the average elastic limit, for the iron columns, and that the actual average fiber stress was 5.3% in excess of the average elastic limit of the material. If we throw out those columns which failed in the end detail this excess is reduced to 3.5%. The crippling load would be expected to exceed the elastic limit.

These tests, it seems to the writer, are of great value, inasmuch as they prove that actual, unsymmetrical, and eccentrically loaded columns do behave as our theory says they will.

Yours very truly,
Clyde T. Morris.

Ohio State University, Columbus, Ohio, Dec. 31, 1907.

Sir: Referring to the series of tests of full-size columns made by Mr. C. P. Buchanan, and so fully presented in your issue of Dec. 26, 1907, I desire to call attention to certain features of these tests and to guard against drawing misleading conclusions from them. They must not be studied as an entire group, but should be classified according to their different characteristics and modes of failure.

First: Tests 5, 6, 7, 8, 10, 11 and 12 all failed in the end details, without crippling or permanent set as a column. Hence these seven tests must be omitted from any study of the series with relation to the ordinary column formulas. They have a very important value of their own, as throwing light on the requirements for proper reinforcement at the ends to assure safe distribution of the stresses from the pin to the several component members of the section. They were discussed from this point of view in a paper by the writer, under the title "A Note on Pin Plates," in the Proceedings of the Engineers' Society of Western Pennsylvania, for March, 1893, and will not be further referred to here.

Second: Of the remaining 12 tests, 4 were of Mr. Strobel's Z-bar section with solid web; 1 was of H-section composed of 4 angles with latticed web; 1 of built channels latticed both sides; and the remaining 6 were of the usual top chord section with cover plate on top and latticed below. Of the latter group 3 were iron, and 3 steel. Of the Z-bar columns 2 were iron and 2 steel. The other two were steel.

Third: The four Z-bar columns, 1, 2, 3 and 4, behaved in the typical manner to be expected of a column; that is to say, they deflected under pressure in a uniform, bow-shaped curve, and passed the yield-point and reached the ultimate without local crippling. After release from the machine they showed permanent curvature in the middle portion, where theoretically the bending moment and resulting fiber-stress are greatest (see Mr. Buchanan's Fig. 1a). All of these tests were of length-ratios in that part of the field where the formulas used for comparison converge and approach the curve of Euler's equation. They agree practically with either of them, and while too few in number to differentiate between the merits of the respective formulas, they tend to confirm the general accuracy of the principal formulas in use.

Fourth: The remaining 8 tests were of latticed forms, 3 iron and 5 steel. Seven of them were of length-ratios bunched between 34 and 46, and only one (No. 9) was of greater length (see Mr. B's. Fig. 20). They all failed by local buckling between lattice points, at pressures less than those computed by the two formulas; except No. 9, which Fig. 9a shows had deflected at right angles to the plane of the web, and more nearly in accord with the normal behavior of a column. The local crippling of the outstanding leg of the angle probably occurred after the point of ultimate resistance had been passed. The others, as will be seen in the several reproduced photographs, failed as above stated by strictly local crippling. They are not sufficiently varied in length and in detail of the latticing to permit of analysis as to the cause of the local weakness, and to suggest the remedy.

Fifth: There is one feature, common to the whole series of tests, which is both interesting and instructive; namely, the surprisingly small deflections exhibited, under crippling and destructive stresses. These tests will have justified themselves even if they teach no other lesson. It will be well for all who have to do with bridges or buildings either in erection or maintenance to study the record presented by Mr. Buchanan, from this point of view, until they thoroughly realize how small a deflection in a previously straight column means danger.

Thos. H. Johnson.

Pittsburg, Pa., Jan. 4, 1908.

[One remark of Mr. Johnson's, namely that local crippling (in Col. 9) probably occurred after the point of ultimate resistance had been passed, is open to doubt. It is much more reasonable, in our opinion, to assume that the distortions after maximum load are of the same kind as those which before failure caused the failure. The proportions of the distorted section would be changed, of course, but the location and general shape would be preserved. Thus, in Col. 9 and more particularly in Col. 13 the distortion after failure (produced by compressing the column considerably after failure in order to show the distortion clearly in the photograph) is in form of a very short kink, a most unexpected kind of failure. We hold there can be no doubt that the weakness which limited the strength of the column was actually contained in these kinked sections, and that the small distortion at ultimate load was substantially of the same form as the large distortion shown in the views; that is, failure was caused by the formation of these short kinks.

Mr. Buchanan's reply to the above letters is as follows:—Ed.]

Sir: If Prof. Morris' table be corrected and extended to include chords 18 and 19 [in which the pins were located 7.64 ins. and 6.95 ins. from under side of top plate], as in the table herewith, his average for iron will be raised and that for steel become about 90%. However, it is not at all clear why any correction should neglect the deflection of the column nor why the elastic limit of the material should be taken, rather than its crippling strength, as the formula is supposed to take care of both the compression and the bending.

Without entering into the analysis of the deduction, I may say that from a study of earlier tests, it was found that unsymmetrical sections with pins in center of web were not affected by the eccentric loading to the extent that was expected and those with pin in center of gravity failed generally in the opposite direction and at lower units than expected. From this it was concluded that as the resistance to bending depended upon the moment of inertia of the transverse section, the greatest resistance would be offered to bending when the pin was located in the axis about which the value of I would be maximum, and this occurs where the pin is midway between above and below the pin balance. The location of pin away from the center of gravity was not intended to produce a moment to balance the moment due to the dead weight of the chord. In the details for use in the bridges, holes for pins were placed at the required distance below this axis. The chord ordered with bridge No. 31, C. & M. V. (No. 14) was punched by the shop foreman without instructions and pins located as in the chords for the structure, so that they were 0.47-in. below the point they should have occupied for test purposes, with the result that it appears as the lowest in the entire group with the exception of No. 5. This latter chord and No. 6 had the ends detailed in accordance with what was then general practice and a method that is still followed; i. e., to take care of the loads shown on stress sheet, whereas the sections provided under requirements limiting minimum thicknesses made the chord good for greater loads. No. 5 had a greater excess of main section than No. 6 and so was affected by the detail to a greater extent. Of course this error was not repeated in any further designs. No. 5 should really be omitted in any consideration of the group.

Test No.	r	v	e	s	Calculated. $\frac{P}{A}$	Actual unit load.	Per cent. of calculated.	Kind of failure.
5	5.55	9.27	0.67	30,700	25,580	23,290	91.2	End de-
6	4.88	8.04	0.54	"	26,020	28,220	108.5	tail.
7	5.69	8.74	0.49	"	27,110	29,210	107.7	"
8	5.69	8.74	0.49	"	27,110	29,900	110.3	"
10	6.34	10.27	0.62	"	26,510	26,200	98.8	"
11	6.33	10.29	0.64	"	26,370	26,230	99.4	"
12	5.65	9.09	0.68	"	25,710	30,770	119.7	"
13	5.80	8.44	0.39	"	27,900	28,200	100.9	Body.
14	4.71	8.04	1.04	"	22,070	26,260	119.0	"
15	6.51	9.86	0.38	"	28,200	30,300	107.5	"
16	6.44	10.36	0.71	39,900	33,930	30,640	90.3	"
18	7.29	11.40	0.64	"	34,000	31,980	94.1	"
19	7.20	11.45	0.00	"	39,900	33,950	85.1	"
Average for iron.....					26,264	27,858	106.3	
Average for body failures							109.1	
Average for steel					35,943	32,190	89.8	

If Prof. Morris' idea, or a similar one, is to be applied to find the probable strength of the column, what is to be done about the transverse deflection? The transverse deflection was about the same as the vertical in four of the tests, was greater in two, and closely approached it in some of the other seven when nearing the maximum. Then, too, the deflections in No. 16, which had the pins located farthest from center of gravity, were *downward* until just before failure; and No. 19, with pins in center of gravity, did not develop any material increase in strength over its duplicate, No. 18, in which the pins were placed 0.64-in. below the center of gravity. On the other hand, the location of pins in center of gravity of middle section of No. 19 resulted in their being 1 in. above the center of gravity of the built-up ends, with a concentration of the applied loads near the top, as shown in the buckled flange, that caused the final deflection to be downward, while the deflections were uniformly upward after the first two loads had been applied.

Taking the pin-friction coefficient as 0.25 and diameter of pin 5 ins., the equivalent lever arm of the resistance to rotation is $\frac{1}{4} \times 2\frac{1}{2} = 0.62$-in., an amount that is only slightly exceeded by five of the sections (with the exception of No. 14, incorrectly bored as noted above). Is it not likely that the resistance to turning offset any tendency to an eccentric distribution of the load to the central portion of the columns and that the manner and point of failure was due to some local characteristic? Mr. Christie (see Trans. Am. Soc. C. E. for August, 1884) in his experiments with rolled columns found that the maximum load was frequently increased by an eccentric application, evidently because of lack of homogeneity in the metal, and this condition is certainly as liable to be found in a built-up member as in a rolled one. The results obtained in the posts of the latticed symmetrical sections are so nearly in accord with those for the chords as to lend color to this view.

Mr. Johnson's suggestion to neglect Nos. 5, 6, 7, 8, 10, 11, and 12 in any study of the series with relation to any column formulae, seems to exclude the bulk of the tests unnecessarily. Granting that No. 5 should not be considered, an inspection of Fig. 20 shows No. 12 as the highest in the entire group of unsymmetrical iron sections, Nos. 7 and 8 above No. 13 (which showed distress at the ends that would have placed it in the group of end failures had the flanges proved equally as good as those in Nos. 7, 8, 12, and 15) and Nos. 10 and 11 higher than No. 14, though, as explained above, the location of pin in No. 14 probably reduced its crippling load.

It may be well to add that all test members were made at the same time that the bridges were constructed, of the same materials, and subject to the same handling and care, *but no greater*, so that the chance of getting an artificially high result that a specially built section might be expected to give was as nearly as possible eliminated. The details were always in line with the best practice of the date of designing and somewhat in advance of the general practice, as experience with the designing forces of the leading companies gave constant evidence. And to the credit of the men forming the many squads that have been engaged upon our work, I am pleased to say that I never found one who was not ready to do everything in his power to improve the details.

Yours truly,
C. P. Buchanan.

Union Station, Pittsburg, Pa., Jan. 13, 1908.

Why the Corps of Engineers Should Continue in Charge of River and Harbor Work.

Sir: I note in your last issue a proposal of the New York Board of Trade to take the river and harbor work of the country out of the hands of the army engineers and place it in the hands of civilian engineers under a proposed Department of Public Works. Permit me to suggest some thoughts upon this subject, based upon an

experience of some years as an army engineer and, more recently, in civil life.

The most menacing defect in our system of government is that there is an ever visible profit to the spoilsmen, the grafters and the political bosses to band into organizations and loot the public treasury, securing the sinews of war for corrupting votes, carrying elections and enriching themselves, all by the use of public funds. On the other hand, the forces working for good government have a difficult fight, with little or no individual reward, and the expenses must be paid from their own funds. Therefore, in this battle, the odds are always in favor of greedy crooks who profit directly, and the conscienceless respectables who profit indirectly.

The political plunderers, bearing different patriotic labels, control practically all of our municipalities. They are Republican in Philadelphia, Democratic in Chicago, Union Labor in San Francisco, etc. They control and rob many of our states. The capitol buildings at Albany and Harrisburg are monuments to their prowess. They are reaching for the National Government, and in some departments there have already reached the looting stage. Fat contracts for engineering works are one of their principal assests. There is such difficulty in making laws to insure fairness and honesty in the awards of contracts that a vast number of honest and leading citizens think that such awards should be left wholly to executive judgment. The City of Philadelphia is robbed annually of millions of dollars by just this method.

The United States Corps of Engineers, recruited as it is under the rigid and fair system of schooling and examination at West Point, is practically beyond political assaults. Its officers are, and always have been, practically out of politics. Their stations are changed so frequently that they have little opportunity for "entangling alliances." They hold an honored position, good for life if they behave, and they are trained to feel a contempt for dishonesty and thievery. The Corps cannot be destroyed piecemeal, and this is a source of enormous strength against political assault.

You cite correctly the fine record that has been made by the Reclamation Service, and it certainly is creditable; but it must be remembered that it is newly organized under a Roosevelt, and that it can be undermined piecemeal. It is not pleasant to say so, but there are crooked engineers and there are political engineers, and it is not hard to conceive that under future political conditions, engineers of that kind can gradually get into the service and eventually undo it.

There is another difference between the work of the Reclamation Service and that of the Corps of Engineers. Each piece of work of the former is for the benefit of a given section and paid for by the inhabitants thereof. These inhabitants will watch the work and fight extravagance and error. The river and harbor work, on the other hand, is paid for from the general treasury and is never paid for by local interests. The latter are benefited by the amount of money spent, whether it is spent wisely or not. Even if the cities paid for their own harbor improvements or paid half of it, the argument would still apply. The scattered few in the country who pay for irrigating their farms cannot be controlled by the political bosses to their own injury as can the denizens of cities. Philadelphians smile at the robbery of their city's coffers and meekly re-elect the executive puppets picked by the contractor-bosses. Of what account would they be to watch their own river and harbor money?

The most potent reason, however, for leaving the river and harbor work where it is, strangely enough, is a military one. The era of perpetual peace has not yet arrived. This country has already had six great wars and may have another at any time.

Our weakness at the outbreak of a war is the lack of soldiers, especially officers, trained in the various departments. No department is more important to success than the Engineer Corps. Each branch requires its special training and its special implements for training. We can and do afford an infantry of reasonable size, and we supply each man with his gun and bayonet, and teach him to use them. We can and must afford to place the great seacoast guns and train the artillerists to use them. We can afford the light batteries, the cavalry sabers and the cavalry horses to train those branches. The signal men, the quartermasters and the commissary officers get ample training with implements of moderate cost and in the routine work of supplying the army during peace. But what country could or would afford to buy bridge material, pile drivers, dredges, snag boats, explosives, steam shovels and all the expensive paraphernalia that would be necessary to give its military engineers the practical experience to make them ready for the multifarious and important emergencies that come with the sudden movement of an army or its other operations?

In practically every army whose engineers are limited to strictly military duty, they rapidly degenerate into telegraph operators and trowel-bayonet experts. The appalling loss of life and capital that occurred while our army during the Civil War was getting licked into shape, is something so stupendous that we should hesitate long before taking any step which lessens the training of our regular officers for the emergency that is sure to arise.

The objection that the army cannot get competent civil engineer assistants, because they cannot be paid salaries above the small pay of the regular officers, is easily remedied. The entire army is now grotesquely underpaid, and the Corps of Engineers, a commission in which is the leading prize for the four years' struggle at West Point, is really paid about the least of all, because they do not get except partially the advantages of quarters and other economies which come to those whose duties are limited to strictly military lines. Congress could, with very good grace, give the army engineers pay somewhat in proportion to their responsibilities.

As it is, however, I think the body of civilian assistants to the Corps of Engineers, in devotion to duty, rectitude and competent ability, will compare favorably with the engineers of the Reclamation Service or any similar body. You will note, too, that all government engineers are paid only moderate salaries.

Another point is that our military engineers must build our forts. These are generally on our most important harbors. The problems of navigation improvement and defense generally interlace and there is economy and absence of friction in having the work in a single office under a single bureau.

The Corps of Engineers has a record for honesty and ability that I believe has never been equaled, and Congress should hesitate long before placing in other hands the work of river and harbor improvement, which is so well conducted where it is now.

Very truly yours,

Cassius E. Gillette.

1021 Land Title Bldg., Philadelphia, Pa., Jan. 30, 1908.

PAY OF ENGINEERS EMPLOYED ON THE PANAMA CANAL.

In the testimony of Col. Goethals before the Senate Committee on Interoceanic Canals, a table was submitted showing officers, employees and salaries in the various departments on the Isthmus. We extract from this table the following list showing the salaries paid the principal officers and employees engaged in engineering work:

1 Chairman*	$11,000	1 draftsman	2,400
1 Commissioner	14,000	6 draftsmen	2,100
1 Commissioner	11,000	14 draftsmen	1,800
2 Commissioners	10,500	17 draftsmen	1,500
1 secy. to Comn.	10,000	3 draftsmen	1,200
1 secy. to Chmn.	3,600	1 draftsman	1,000
3 secys. to Comrs.	2,400	1 tracer	1,200
1 chief of Division of Material and Supplies	10,000	1 tracer	900
1 disbursing officer.	7,860	9 instrument men.	2,100
1 asst. disbursing officer	4,000	11 transitmen	1,800
1 exmr. of accts.	4,500	22 levelmen	1,500
1 secy. to chief Division of Material and Supplies	2,100	16 levelmen	1,200
6 division engrs.	7,500	4 rodmen	1,200
1 division engineer.	6,600	24 rodmen	1,000
1 division engineer.	6,000	5 rodmen	900
2 division engineers	5,000	1 supt. of motive power	7,500
2 division engineers	4,200	1 supt. of transportation	5,000
1 division engineer.	3,500	1 asst. supt. of transportation	3,000
1 office engineer.	4,800	1 master car bldr.	3,600
1 asst. engineer.	4,000	1 train master.	3,000
8 asst. engineers.	3,000	1 train master.	2,700
4 asst. engineers.	2,700	24 yard masters.	2,520
3 asst. engineers.	2,400	1 superintendent	4,200
2 designing engrs.	6,000	1 superintendent	4,000
2 designing engrs.	4,200	9 superintendents	3,900
1 designing engr.	3,600	2 superintendents	3,300
1 mechanical engr.	5,000	16 superintendents	3,000
1 resident engr.	4,000	8 superintendents	2,700
5 resident engrs.	3,600	2 superintendents	2,400
1 resident engr.	3,300	1 superintendent	2,100
1 asst. mgr. Labor and Quarters.	4,000	1 supervisor	3,600
1 master builder.	6,000	1 supervisor	3,000
1 asst. master bldr.	3,600	1 supervisor	2,640
1 architect	3,600	6 supervisors	2,400
1 draftsman	3,000	12 supervisors	2,100
1 draftsman	2,700	2 supervisors	1,800
		4 supervisors	1,500
		4 supervisors	1,200

PRIVILEGES GIVEN BY THE ISTHMIAN CANAL COMMISSION TO ITS EMPLOYEES IN ADDITION TO SALARY.

Quarters.—Bachelors.—A bed in a room with one or two or three others, according to salary received.

Married men (employed prior to Jan. 1, 1908).—Housekeeping quarters consisting of 4 rooms, including kitchen, and upward, according to salary received. Fuel, electric lights and water free.

The allowance of furniture for families of employees receiving less than $400 per month is fixed as follows: One range, 1 double bed, 2 pillows, 2 kitchen chairs, 6 dining chairs, 1 chiffonier, 2 center tables, 1 mosquito bar, 1 refrigerator, 1 double mattress, 1 kitchen table, 1 dining table, 1 sideboard, 1 dresser, 1 bedroom mat, 3 wicker rockers.

For employees receiving $400 per month or over the following additional articles are authorized: Three dining chairs, 1 chiffonier, 2 towel racks, 1 parlor desk, 2 parlor chairs, 1 porch swing, 1 serving table, 1 dresser, 1 bedroom mat, 1 parlor wicker rocker, 1 morris chair, 1 porch double seat.

The issue of beds, mattresses, and mosquito bars in excess of this allowance is authorized to families with children.

Medical attendance and medicine free to employees. Medicines free to families of employees. Visits of physicians to families, $1 each.

Sick leaves with pay given to all employees on the Gold Roll amounting to 15 days for each six months, cumulative, not to exceed 30 days in any one year. Meritorious injury leave with pay in case of accident through no fault of employee not to exceed 30 days in one year, to all employees.

One round-trip pass on the Panama Railroad for each employee on the Gold Roll per month, for which the Commission pays half the regular rates. Half-rate tickets on the Panama Railroad to employees' families. Reduced rates on the ships of the Panama Railroad to employees and families when traveling on leave. (The rate at present is $20). To all monthly employees on the Gold Roll, six weeks' leave per year with pay.

*The full salary of the Chairman is $15,000 a year and each Commissioner receives $14,000; but those who are army officers have their regular pay deducted from this salary.

NINETEEN OPINIONS ON NEEDED CHANGES IN ENGINEERING EDUCATION.

At a special meeting of the American Institute of Electrical Engineers held in New York City, Jan. 24, 1908, and entirely given up to discussions of better electrical engineering education, Dr. C. P. Steinmetz advocated a reduction of the number of studies in the curricula of the schools of the country, and a corresponding increase in the thoroughness of the fundamental studies, such as chemistry, physics and general culture subjects. He advocated carrying each and all subjects in such a way that they would be studied during the entire college course in a progressive manner. He claimed that the present system filled the student's head with a mass of details and left his grasp on the principles weak and his capacity for logical reasoning impaired or underdeveloped.

C. F. Scott (Consulting Engineer, Westinghouse Electric & Manufacturing Co.) presented a short introduction to an abstract of Institute papers on Engineering Education. In this he divided the instruction into practical, scientific and cultural and stated that the discussions should show the desirable characteristics of an acceptable graduate and those arrangements of curricula that will produce the desired results. In this introduction the much discussed "concentric method" of Professor v. Karapetoff, of Cornell University, was held to be the nearest approach to a solution of the present problem. This proposed method starts in the freshman year with about 45% of the time devoted to general culture, 45% to non-electrical engineering and 5% each to electrical engineering and to auxiliary sciences. The amount of time devoted to the first two classes decreases during the four years allotted, while that for the latter two groups increases until at the end of the senior year the culture and non-electrical engineering studies each occupy 5% of the whole time of the student and electrical engineering and auxiliary science each 45%.

An epitome of the expressions of those who discussed these two papers forms an interesting comparison of the contemporary opinion of men of recognized standing:

Loyal A. Osborne (Works Manager, Westinghouse Electric & Mfg. Co.).—Of the engineering graduates who seek employment 50% had better not have taken up the profession. Engineers are too narrow because their early education did not stimulate powers of generalization.

H. E. Clifford (Professor of Theoretical Electricity, Mass. Institute of Technology).—Engineering curricula should not become standardized if progress in teaching methods is to result. The number of studies given needs to be reduced and all experimental work should be in the nature of true engineering problems. The gradual elimination of unfitted students must be maintained throughout the courses.

Francis B. Crocker (Professor of Electrical Engineering, Columbia University).—Professor Karapetoff's "concentric method" has one fatal educational difficulty. It would make engineers of all students who enter the courses. There is no provision for elimination of unsuited men from the courses.

H. W. Buck (Electrical Engineer, Niagara Falls Power Co.).—Experiment should precede theory. The practical meaning and application of all mathematical processes must be emphasized or the proper conception of mathematics is lost.

W. S. Franklin (Professor of Physics, Lehigh University).—All courses in physics and chemistry should be more progressive. The fundamentals should be presented in brief courses so as to be readily and thoroughly

grasped. The technology of the subject should afterwards be added in amplified courses.

L. B. Stillwell (Consulting Engineer, New York City).—The ideal education cannot be too broad. The aim should be to educate and train rather than to inform. The engineering training might well follow a collegiate course.

A. F. Ganz (Professor of Electrical Engineering, Stevens Institute of Technology).—The college should teach those things which the engineer needs but cannot readily obtain in practice. Laboratory practice and theoretical teaching should go hand in hand; one should not precede the other. The student needs to learn sources of information rather than burdening his mind with data.

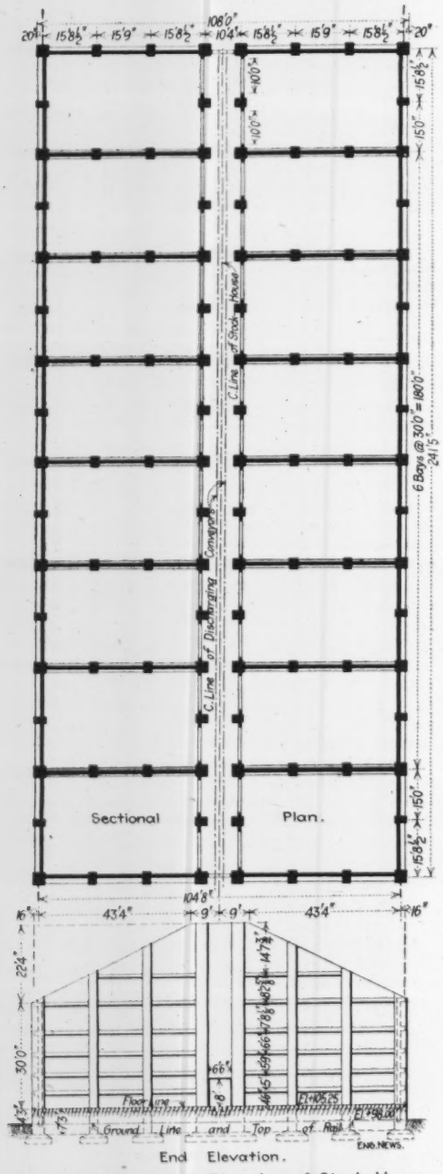


Fig. 1. Plan and End Elevation of Stock House.

J. G. White (President, J. G. White & Co., New York City).—The courses should lead to a good general knowledge of mathematics, physics and chemistry. There should be more text books, fewer lectures and no elective studies.

L. A. Ferguson (Vice-President, Chicago Edison Co.).—The proper education should develop an analytical mind able to distinguish the right and the wrong of engineering. There is great danger of "faculty inbreeding." The instructing staff of a college should not be all alumni of that college, but should contain men from every section.

Samuel Sheldon (Professor of Physics and Electrical Engineering, Brooklyn Polytechnic Institute).—The tuition of students should be raised to provide adequate salaries for good engineering instructors. The funds of philanthropists should pay the tuition of selected students who are unable to meet the increase. The advantages claimed for the "concentric method" are badly needed but they can be obtained by less radical schemes. This method as outlined leads to hiatus and inefficiency.

Percy H. Thomas (Consulting Engineer, Thomas & Neall, New York City and Boston).—In 10 or 15 years the engineer should acquire familiarity with (1) the laws of nature and the properties of materials, (2) mathematical methods of working, (3) results of experience and sources of information, (4) knowledge of different practice in different localities. Only the first two are properly to be taught in colleges, with very little about specific design, determination of commercial constants, advantages of various makes of apparatus, etc.

Wm. L. Robb (Professor of Electrical Engineering, Rensselaer Polytechnic Institute).—Eventually culture subjects will pass from the curricula of technical schools and these will be on the same plane as medical and law colleges with the highest of entrance requirements.

C. O. Mailloux (Consulting Engineer, New York City).—The use of text books and lectures should be combined. The text books should be small and should thoroughly cover the few fundamental principles. Use applied mathematics more and pure mathematics less.

D. C. Jackson (Head of Department of Electrical Engineering, Mass. Institute of Technology).—All general studies should be followed through a man's course and not ended in any year to allow a new and different course to be started.

Philip Torchio (Chief Engineer, New York Edison Co.).—Present technical schools seem to aim to turn out two classes of engineers—immediately practical men and those fitted for broader fields of usefulness though less practical at the start.

H. G. Stott (Superintendent of Motive Power, Interborough Rapid Transit Co., New York City).—The preparatory school studies need adjustment as much as college courses. Many present troubles will be eliminated by raising the entrance requirements. Colleges should teach the underlying principles and not engineering practice.

SELF-SUSTAINING REINFORCEMENT OF STRUCTURAL SHAPES IN A CEMENT STOCK HOUSE.

A large stock house for storing finished cement, built last fall as part of the new plant of the Vulcan Portland Cement Co., Longue Pointe, near Montreal, Que., embodies an interesting example of concrete reinforcement so designed as to make a self-sustaining steel framework before the concrete is placed. Such arrangement of the reinforcement affords the considerable advantages of definite, immovable placement of the steel, and elimination of a large amount of scaffolding otherwise necessary to support and brace the forms for the concrete. Primarily the sections of steel required for reinforcing purposes alone are so small that it is difficult or at least uneconomical to design them of structural shape steel suitable for riveting or bolting together in a framework. In the present case, however, the design is very heavy, because the walls have to resist the thrust produced by a 30-ft. depth of cement; accordingly, the steel sections are substantial and make up a strong framework. The principal elements of the structure are briefly described in the following:

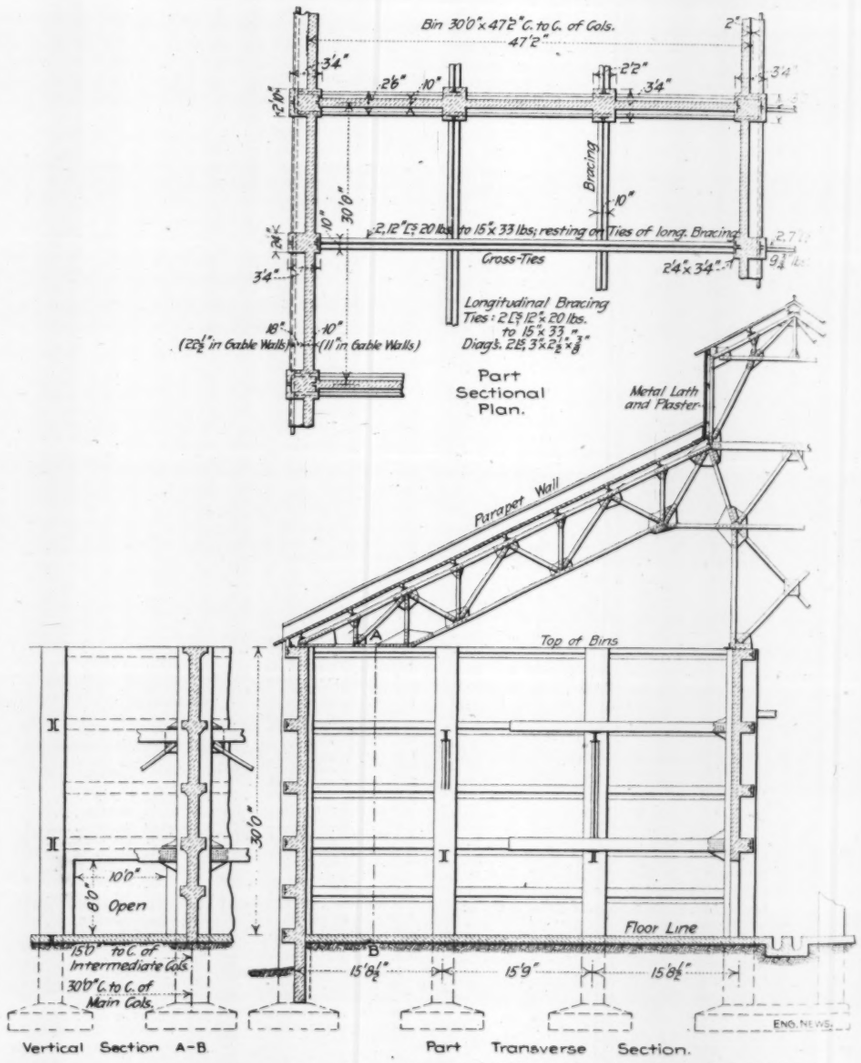


FIG. 2. SECTIONS THROUGH STOCK HOUSE, SHOWING TYPICAL CONSTRUCTION.

A ground plan and an end elevation of the building are given in Fig. 1. The building has a storage capacity of about 240,000 bbls. of cement. Its floor area of 108 ft. × 245 ft. 3 ins. (external dimensions) is divided into two parts by a central passage 10 ft. wide running lengthwise through the building. The two side bays are divided by transverse partition walls spaced generally 30 ft. on centers into bins measuring 30 ft. by 47 ft. 2 ins. There are intermediate

columns in the walls, however, about 15 ft. apart, and these are tied together by steel members running through the bins. The beam and slab wall construction spans the intervals between columns and sustains the thrust of the cement on the 15-ft. panel. The partition walls and the two longitudinal walls which form the central passage are all carried to the height of the eaves; only the gable walls rise above this level.

The roof is of plank, covered with composition, carried on steel trusses and purlins. The trusses are spaced 15 ft. on centers, the same as the columns. The roof surface is pitched 6 ins. per ft. from each of the long sides to a central ridge; a monitor extends along the full length of the ridge. Two longitudinal 12-in. screw conveyors in the monitor, supplied by a transverse belt-conveyor in one of the end bays, receive the cement from the grinding mill and distribute it among the different bins and the central passage. It is not intended however to keep the cement in the several bins separate, or to fill only to the eave level. The walls must therefore be able to carry a surcharge sloping up from the eaves on the angle of repose to the peak, that is, a load following practically the outline of the main roof. Four pairs of horizontal and vertical diagonal ties stiffen the upper parts of the gable walls against this pressure. The floor of the house is of plain concrete, 9 ins. thick, resting on an earth filling. Running along the central passage are two conveyor troughs molded in the floor, for screw conveyors. Openings 10 × 8 ft. in the walls communicate with the bins.

The arrangement of walls, partitions, roof construction, and steel ties and bracing in the bins is shown by Fig. 2. The columns are rectangular, and maintain the same dimensions for their full height; the smallest columns are 24 × 40 ins., the largest 40 × 45 ins. in section, the larger dimension being transverse to the wall in every case. They rest on reinforced-concrete footings whose bottom surface is 6 ft. below ground level. The wall between columns consists of horizontal beams spaced 4½ ft. to 8¼ ft. apart vertically, and a 10-in. slab (11-in. slab in the gable walls) spanning vertically between the beams. The partition walls, which may be subjected to thrust from either side, have the beams arranged symmetrical to the wall slab, i. e., projecting on either side of the slab. However, these are not so much reinforced-concrete beams as steel girders whose flanges are tied together by steel separators and bolts and a concrete filling. The outer walls as well as the walls separating the bins from the central passage will be subjected to thrust from one side only, the latter for the reason that, although the central passage is also to be filled with cement, the conveyors will always fill the passage after, and empty it before, the adjoining bins. The beams in these walls are placed on the side away from the thrust, or on the passage side; this enables them to be designed as T-beams, making use of part of the slab concrete for their compression flanges.

The reinforcement is exhibited by typical detail sketches in Fig. 3.

The column reinforcement or core consists in every case of four angles grouped in an I-section by batten-plates (no latticing). Where they rest on the concrete foundation blocks they have regularly detailed footings, and the connections between columns and roof trusses, and between columns and tie-members as well as beam cores, are in all cases riveted joints and are fully detailed to carry the weight of the framework and sustain all thrusts and incidental erection stresses. The angles used for the column reinforcement are 6 × 4 × 7/16-in. in the intermediate columns and 6 × 4 × ⅜-in. in all others. They are arranged in pairs with the longer legs outstanding. In the partition columns the two pairs have the backs of angles facing each other, but in the other columns the short legs are turned in the same direction. Binders of ⅜-in. rod are wound around the column steel every 18 ins. vertically. As already noted, the beams of the partition walls are composed of concrete filling between two steel channels, one on either side of the wall, spaced 30 ins. back to back, flanges in. These channels are riveted to the outstanding legs of the column angles. Through-bolts set at intervals tie the two channels of a pair together.

The beams of the longitudinal walls and the gable walls have as their core and principal reinforcement a pair of horizontal steel angles, short legs riveted together, set near the outer face of the beam and riveted to the column angles. These angles are 6 × 4 × 5/8-in. in the lower section and decrease to 3 × 2½ × ⅜-in. in the upper beams. The thickness of concrete outside the angles is 1 to 1½ ins. The width of beam (vertical dimension) is 14 ins. in most cases. The depth or horizontal dimension of the beam is kept constant at 18 ins. outside of the slab (22½ ins. in the gable walls), or 28 ins. including the slab (33½ ins. in the gable walls). These beams have the following additional reinforcement placed at the time the concreting is being done: Reverse reinforcement at the supports, provided by four ¾-in. round rods 6 ft. long placed near the inner face of the wall at the columns; and stirrups of varying number and spacing to give the necessary web reinforcement and bond the angles firmly to the concrete. Half of the stirrups are of U-shape, their bent ends being hooked through holes punched in the vertical legs of the angles; the others are single rods hooked into holes in the horizontal legs of the angles. The number of stirrups in a single beam varies from 24 to 60.

The slab reinforcement is, like the beams, symmetrical in the partition walls and unsymmetrical in the longitudinal and gable walls. In the former it consists of seven ¾-in. round rods per panel on each side of the slab, placed vertically, extending from bottom to top of wall; in the other beams it comprises eight continuous ¾-in. rods near the outer face of the slab, and the same number of ½-in. rods 3½ ft. long near the inner face of the wall at each beam for reverse reinforcement.

The arrangement of the interior steel bracing in the bins is fully shown by the drawings. The two upper of the three tiers of ties transverse to the building rest on the ties running in the other direction, the latter being trussed to prevent sagging. The lower set of transverse ties is embedded in the floor and its top edge is flush with the floor level.

ELEMENTS OF DESIGN.—Several factors cooperated in bringing about the adoption of the riveted, self-sustaining type of reinforcement. Heavy stresses had to be provided for, not only shears and bending moments but also tension in every wall; the latter particularly required the steelwork to be joined by strong connections at the corners, and with rod reinforcement this would have called for much bending and forging, giving expensive and uncertain connections. The available labor was of poor quality, which again made it desirable to eliminate skilled field work as far as possible. Further, as the construction was started very late in the season and of necessity therefore had to be carried on under unfavorable weather conditions, it was thought still more necessary to eliminate as far as possible the need for skilled and careful labor in the field. The building having high walls without intermediate floors, special trouble was to be anticipated in alining and bracing the wall forms, which difficulty was avoided by the steel skeleton design, detailed so that the forms could be attached to and braced by the steel framework. The design shown proved to meet all these needs very satisfactorily.

The cement pressure in the bins was calculated on the basis of the following: Angle of repose constant, 15°; weight of cement, 100 lbs. per cu. ft.; coefficient of friction between cement and walls, 0.55 for the plain side of the walls and somewhat higher for the ribbed side. But the friction was considered only in calculating the vertical loads on the walls, columns and foundations; the lateral pressure was calculated without reference to the friction, by Rankine's formula for surcharged walls.

It was believed that, as experiments showed the lateral pressure to be greatest just after the cement has come to rest, the calculated pressure would be higher than that actually realized when the filling proceeds so slowly as to require several days to fill the bin, as here. For this reason higher working stresses were used than would otherwise have been taken. The permissible compression in the concrete was taken as 700 to 800 lbs. per sq. in., the shear at 70 lbs. per sq. in., the tension in the steel at 18,000 to 20,000 lbs. per sq. in., and the ratio of moduli at 12.

The beams and slabs were designed as partially continuous over the supports, using $\frac{Wl}{10}$

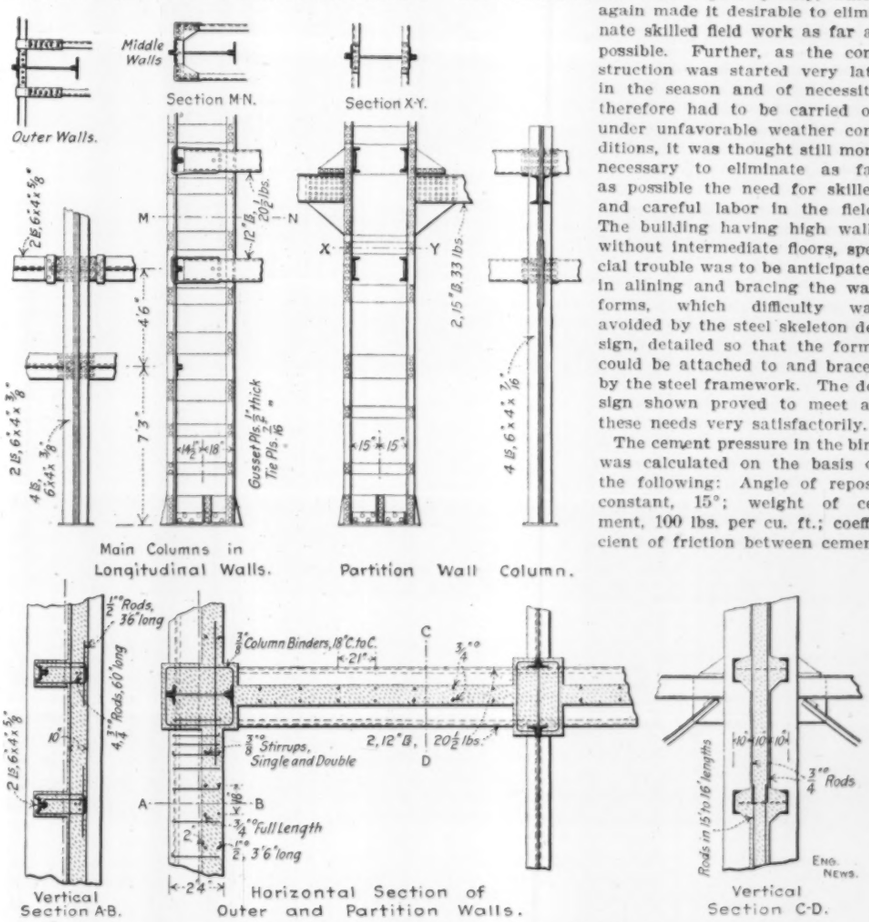


FIG. 3. DETAILS OF COLUMNS AND WALLS.

$\frac{Wl}{20}$ and —— as the moments at center and at support. The partition beams were considered to be steel girders, as previously remarked, so that the concrete filling was required to carry the web stresses only. The beams in all other walls were calculated as T-beams, using a width of two-fifths of the shortest adjoining span as total width of compression flange.

CONSTRUCTION.—The concrete used for the reinforced footings was 1:3:5, with broken stone of 2 ins. or smaller. The concrete for columns and walls was mixed 1:2:4, the stone being broken to a maximum size of ¾ in., and screened to remove the dust. The stone, limestone, was quarried and crushed on the job, and was of fair quality, but it contained much dust, which the water turned into a plastic clay, for which reason the stone for columns and walls was screened.

The exposed steel (ties and outside faces of partition-beam channels) was given two coats of paint in the shop; all other steel which was to be embedded in the concrete was left unpainted.

The riveted steel frame was first erected without the roof framing; pneumatic hammers were used for most of the riveting. The anchor bolts for the columns were set in the footing concrete, but inside of 2-in. pipe, so as to allow for inaccuracies in the footings and the metalwork. The pipes were later filled with cement grout. After the riveted steel was erected two guy derricks were set abreast on this frame, one on each side of the central passage. These handled the forms and the concrete buckets on three-fourths of the structure, while for the remainder a hoisting tower and a stiff-leg derrick were put up, in order to hasten the work through before the severe cold.

Forms for the columns were made in front and back sections, extending inward to the slab on one side and to the outer edges of the beams on the other. Forms for the walls were made as sketched in Fig. 4, in sections covering an entire wall panel between columns and beams of such construction that each section was supported by the steel frame and the lower sections could be removed before the upper ones. In the forms for the longitudinal and gable walls (upper sketch in Fig. 4), where successive sections of the forms are in direct contact; this was done as follows: The vertical stud frames, four per section, which carry the lagging, bear downwardly projecting bracket pieces which hook over the upturned leg of the steel angle forming the reinforcement of the lower beam. This holds the lower edge of the form against outward shifting, and holds it up to level; at the same time the form is held outward by a wedge which bears on the outer face of the studs at the joint with the studs of the form-section below. The upper edge of the form bears against the steel angle of the beam reinforcement by a narrow block and is held inward by the wedge-connection with the form-section above. To remove a form-section the bracket pieces in its lower part are detached by driving out the pins which hold them to the stud frames; this frees the lower edge of the section. The upper edge is freed by driving out the wedges at the upper edge. The form can then be lifted out freely. The sides of columns and beams were battered ⅜ in. per ft. to make the forms draw clear without binding.

The forms were made of 3 × 3 in. studs and 1½-in. hemlock lagging, surfaced on one side and matched. The inner face of the form was oiled before using. Forms were at first provided for one complete quarter section of the building, and it was the intention to re-use these till the job was completed. On account of weather conditions, it was afterward decided to provide some additional forms and start operations at the other end of the structure. As a result of this the first set of forms was actually used only three times.

The forms were built in a temporary carpenter shed erected near the building, equipped with motor-driven saw, etc. The rod reinforcement required little work, as the rods were ordered to length. A 1⅛-in. rod shear did the necessary cutting, and the cold bending of stirrups and column binders was done by a simple hand bender attached to the bench.

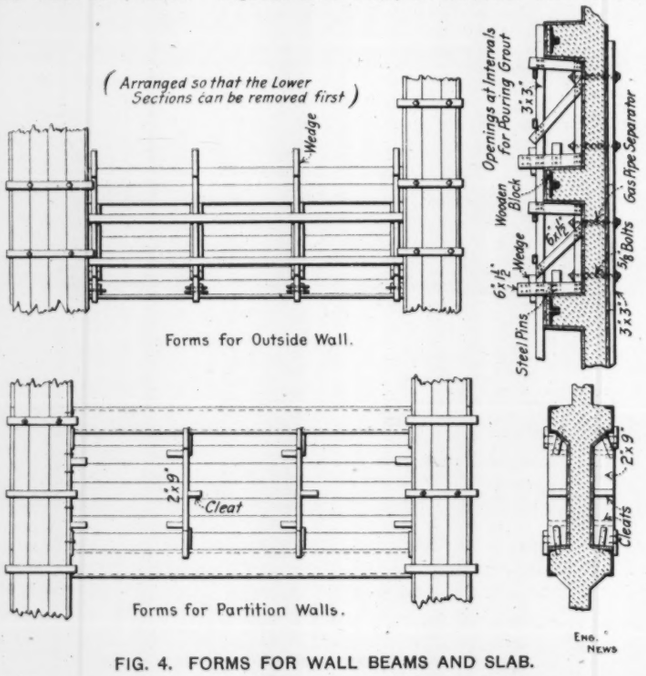


FIG. 4. FORMS FOR WALL BEAMS AND SLAB.

The form sections for the outside face of wall were generally erected complete to the top and supported on the steel frame before any concreting was done. The reinforcing rods were then placed and held in position by means of wooden strips, provided with notches or holes for the rods, nailed to the outside forms. The inside forms were then erected section by section as the concreting progressed, and were held in position by pipe separators and bolts passing through the outside forms. The wooden spacers for the reinforcing rods were removed before the concreting reached them.

The concreting proceeded from bottom to top of a single panel, without reference to the adjoining panels. Construction joints were made on the line of the column webs, and in the wall work, when necessary, on the horizontal center line of a beam.

The concrete was mixed by two Ransome mixers on trucks and located on each side of the building between successive positions of the derricks. Two 14 cu. ft. contractors' tipping buckets handled by the guy derricks transferred the concrete from the mixers to the walls. When a section within the reach of the booms was completed, including the moving of forms, the derricks were moved forward.

The instructions issued to the field force provided that the maximum dump of the concrete should not exceed one panel-height, and further, that construction joints would be permitted only at the center line of beams and columns. Concreting was permitted whenever the temperature was above 15° F. When below 50° F. the stone, sand and water were heated and steam was discharged into the mixer. All fresh concrete less than 12 hrs. old was protected from freezing.

The principal quantities involved, exclusive of packing houses, are:

Concrete, not including floors.....3,570 cu. yds.
Fabricated steel, not including roof....510 tons.
Steel rods .....161 tons.

Work on the footings was started Sept. 2, and concreting of walls was completed Nov. 28, 1907.

All steel was inspected at the mills, and each carload of cement at the job was sampled and tested in accordance with the specifications of the Canadian Society of Civil Engineers.

The stock house is continued at each end by an enclosed steel shed used as packing house. These sheds are as wide as the stock house, 108 ft., and are each 30 ft. long, so that the total length of the structure is about 305 ft. They consist of steel columns and roof-trusses, footing on concrete foundation walls, roofed with composition on boards (like the stock house), and enclosed with walls of trussed metal lath clipped to the steel girts and plastered both sides with cement.

The structural steel frame for the stock house and packing houses was fabricated by the Structural Steel Co., of Montreal, and was delivered at the site complete within six weeks after the order was placed. All other work was done by the engineers by day labor.

The building was designed and erected by W. S. Barstow & Co., Engineers, of New York City. Mr. A. Jordahl conducted the concrete work for them, and to him we are indebted for the notes and drawings here given.

THE ROOSEVELT DAM has been carried to a height which makes it safe against floods, according to a statement received by the Washington office of the U. S. Reclamation Service from Mr. L. C. Hill, Supervising Engineer in charge of the Salt River Irrigation Project, Arizona. The contractor for this work, Mr. J. M. O'Rourke, of Galveston, Tex., laid the first stone of the dam in September, 1906. Mr. Hill is located at Phoenix, Ariz.

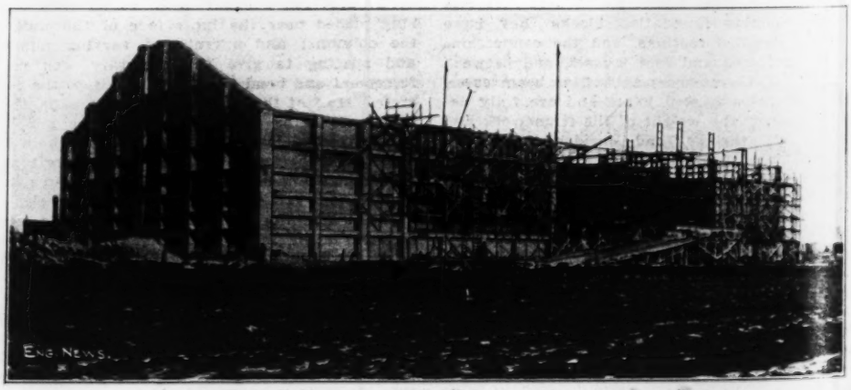


FIG. 5. A VIEW OF THE CEMENT STOCK HOUSE WHEN PARTLY COMPLETED.

HOW LARGE A WATER SUPPLY CAN BE DRAWN FROM CROTON DRAINAGE AREA, NEW YORK CITY?

By FRED D. FLINN,* M. Am. Soc. C. E.

Probably few if any other drainage areas in America have been so thoroughly studied as those of the Croton and Sudbury Rivers. For two generations the former has furnished the larger part of the water consumed by New York, and, from the latter, Boston drew the major part of its supply for 30 years. Rainfall and runoff records for these streams have been kept for longer periods consecutively than for any other streams similarly used. Consequently, many lessons of general value may be deduced. Quite recently, in connection with the studies for New York's new water supply from the Catskill Mountains, and the filling for the first time of the new Croton Reservoir, completed in 1905, some interesting computations were made as to the greatest possible daily supply which could be drawn from the Croton drainage area and the cost of the necessary works.

The sight of water pouring over the 1,000-ft. waste weir of the New Croton dam, at depths varying from a few hundredths to 1 ft., since Nov. 6, 1907, with all the reservoirs full, has led some laymen, and even engineers imperfectly informed, to believe that here was a great waste of water which could be readily conserved. If this water could be stored, are new sources of supply immediately needed? Since similar considerations arise in connection with many water supplies, the facts and conclusions in this case have more than local interest.

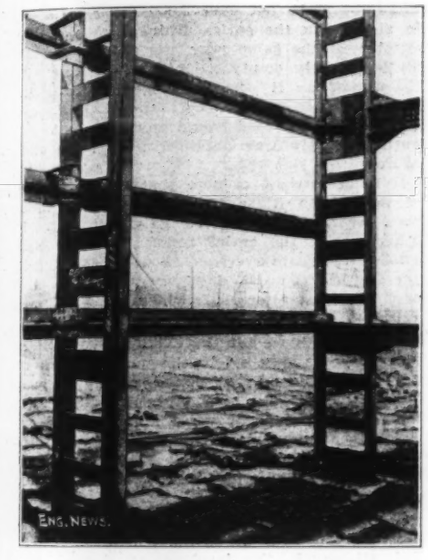


Fig. 6. Two Outside Wall Columns.

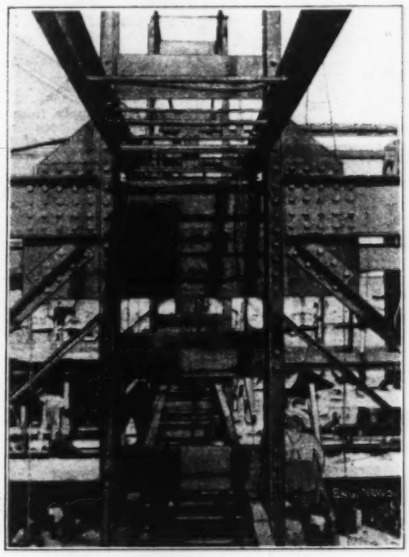


Fig. 7. Looking Along a Partition Wall.

FIGS. 6 AND 7. TWO VIEWS OF THE RIVETED REINFORCEMENT.

On the two drainage areas mentioned it was not feasible to secure sufficient storage capacity for maximum development by one great reservoir at the point of diversion of the stream. Consequently, a number of reservoirs have been constructed. On the Sudbury drainage area there are now eight reservoirs, with a total capacity of 13,616,000,000 gals., giving a storage of 181,000,000 gals. per sq. mi. Considering the individual reservoirs, the storage per square mile for the portion of the drainage area tributary to one reservoir is 328,000,000 gals. With this development, the Sudbury drainage area is reckoned to be capable of yielding safely a supply at the rate of 53,000,000 gals. daily, or 700,000 gals. per sq. mi. per day through such critical series of dry years as have actually occurred. Development of this area is considered to have been carried to the economical limit, and the supply has been supplemented within late years by the extensive works on the Nashua River, well known to readers of this journal through articles on the Wachusett dam and reservoir.

On the Croton drainage area nine reservoirs have been built from time to time, and six natural ponds improved for storage purposes. Another reservoir, the Cross River, has just been completed and is being filled, while the eleventh reservoir, known by the name of Croton Falls, is under construction. The total available capacity of these reservoirs and ponds, including flashboard storage, will be 104,530,000 gals., equivalent to about 290,000,000 gals. per sq. mi. of drainage area. For part of the drainage area tributary to a single reservoir, however, the storage capacity reaches 449,000,000 gals. per sq. mi.

With this development it has been computed that the Croton River can just furnish 336,000,000 gals. daily through a critical series of dry years no worse than has actually occurred during the past 40 years, of which we have record. This is equivalent to 930,000 gals. per sq. mi. per day, which is greater by 230,000 gals. than the corresponding quantity for the Sudbury. This is a higher value than eminent engineers have heretofore given as the safe yield of the Croton, and in view of the probability that the future may have in store a dryer series of years than have occurred in the last 40, it must be evident that this figure of 336,000,000 gals. daily has, as it were, no factor of safety. The average rainfall for 30 years on the Sudbury has been 46.1 ins., while 40 years on the Croton have given an average of 49.2 ins. The average percentage of rainfall collected has been 48.6 on the Sudbury and 48.9 on the Croton.

When the construction of Cross River and Croton Falls reservoirs was determined by the Croton Aqueduct Commission, upon the recommendation of Mr. J. Waldo Smith, M. Am. Soc. C. E., who was then their Chief Engineer, and Prof. William H. Burr, M. Am. Soc. C. E., Consulting Engineer, extensive computations were made by the writer which indicated clearly that the limit of economical development would be reached, if not slightly exceeded, by the construction of these two reservoirs. Indeed, Mr. John R. Freeman, M. Am. Soc. C. E., in his report of 1900, had questioned the economy of building Cross River Reservoir if another source of supply could be secured promptly. Delay in getting new sources, and the time inevitably necessary for building the great works to bring the new water to the city, were chief reasons for recommending Cross River and Croton Falls reservoirs. These could be more quickly constructed than the great new system of works, but even their usefulness in the emergency was clearly dependent upon the occurrence of years of somewhat more than average rainfall at their completion. They were recommended only as an emergency measure, because the beginning of an additional supply had been deferred.

During the computations in 1905, a diagram was drawn showing the natural annual flow of Croton River, and it immediately appeared that the 36 years from 1869 to 1904, inclusive, fell naturally into two 18-year periods, the earlier of which was strikingly dryer than the second, the average natural annual flow of the river for the two periods being, respectively, 127,000,000,000 gals. and 165,000,000,000 gals., the second being 30% greater than the former. Many hydraulic projects are designed and built upon the basis of much shorter records than 18 years, but this history on the Croton drainage area shows clearly how misleading the records of even so long a period as 18 years may be, leading either to excessively high or unduly low assumptions, according to the character of the period covered by the record and the nature of the work contemplated. Furthermore, the effect of the distribution of the precipitation throughout any given year is strikingly illustrated by the record of the year 1907. During the first eight months, the rainfall amounted to only 28.1 ins., which was 5 ins. below the average. About the middle of September, however, a series of storms of intense precipitation occurred, which rapidly filled the streams and reservoirs, and, at the end of the year, had raised the total precipitation to 59.6 ins., which was exceeded by only two other years during the past forty. For the last four months of 1907 the rainfall was 18% higher than in any other four months in the 40-year period, and for September and October, 10% higher than for any other two months in the 40 years.

In December, 1907, a study was made of the additional reservoir capacity required to prevent all flow of water over the spillway of the New Croton Dam, assuming such flows in the river as have occurred during the following periods: 18 dry years, 1869 to 1886; 18 wet years, 1887 to 1904; 40 years, 1868 to 1907; also the possible increase to the daily supply to be obtained from the Croton drainage area by building such additional reservoirs, together with a very rough approximation of the probable cost.

A summary of the results obtained follows. Attention is also called to diagrams, Figs. 1 and 2, showing graphically the most striking features of the results. The reservoir capacity would have to be nearly four times as great as at present, and the daily supply would be increased only 14%, to prevent waste throughout the 40-year period. To conserve all the water during the wet 18-year period, the reservoir capacity would have to be nearly trebled, while adding 30% to the average supply drawn daily. This would necessitate an additional aqueduct of an average daily capacity of about 90,000,000 gals., the two existing Croton aqueducts having a combined capacity of only 380,000,000 gals. Long before such reservoirs could be built, the city would need a greater supply than this. But dryer years are sure to come.

In making this study the same method has been employed as was used by Mr. John R. Freeman in his report to the Comptroller on New York's water supply, in 1900. However, we now have rainfall and runoff records for eight years longer than were available at that time, and they have shown that the runoff, instead of decreasing, as was naturally to be expected, has increased, and even for the present year is considerably above the average for the entire 40-year period.

The average yield for the entire 40-year period, assuming a water supply as at the completion and filling of the Croton Falls Reservoir, is 1,100,400 gals. per sq. mi. per day, or 402,000,000 gals. per day from the entire drainage area. The total storage that would be necessary to obtain this yield would be 409,000,000,000 gals. There is an absurdity in this deduction in that it is physically impossible to provide such a storage as this without greatly increasing the percentage of water surface on the

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drainage area. A correction for the evaporation from this added water surface is necessary. A storage of 409,000,000,000 gals. would give a water surface of about 20% of the total drainage area.

It may be assumed as approximately true that a mass curve plotted for a greater water surface than 5.36%, that which will exist at the completion of the Croton Falls Reservoir, would show the necessity for the same total capacity of storage, in order to get the entire yield, but the average draft for the period would, of course, be lower. An inspection of a mass curve showing lines for several percentages of water surface will show the truth of this.

The question then becomes: how much will a given increase in percentage of water surface reduce the average draft? We can, of course, get this directly by building up a mass curve for the desired percentage of water surface, but it is possible to get the correction by the use of the evaporation tables on page 195 of the Freeman report. From Column 22 of that table, it appears that the loss from substituting 1 sq. mi. of water surface for 1 sq. mi. of land surface would average 699,000 gals. per day. For a 20% water surface, then, we would have 20% of 360.44 (total drainage area) — 19.32 (area of water surfaces after filling of Croton Falls Reservoir), or 52.8 sq. mi. changed from land to water by the increase of the water surface from 5.36% to 20%. The decrease in draft would then be 0.699 × 52.8, or 37,000,000 gals. per day. Therefore, the draft that could be obtained from the Croton drainage area, assuming a 20% water surface, would be 402,000,000 —37,000,000, or 365,000,000 gals. per day.

Another factor now enters into the problem, due to the fact that, especially with any such extraordinary development as this, the percentage of water surface cannot properly be taken as that with the reservoirs full, because we have assumed at the beginning that they were all full only once in 40 years. A study of the mass curve shows that the average condition of the reservoirs for the 40-year period was almost exactly half full; therefore, a curve of percentage of water surface, assuming the reservoirs half full, would be the low limit. Bearing in mind that the increase in the evaporation when the reservoirs are above the average condition is more than the decrease when they are below it, it was decided that the condition more exactly approximating actual conditions would be obtained by assuming the water surface to be that of the reservoirs two-thirds full.

Going over the computations again, we find that the water surface for 409,000,000,000 gals. is about 13%. Then 13% of 360.44 — 19.32 = 27.54 sq. mi.; 27.54 × 0.699 = 19,000,000 gals. per day as the total that should be deducted from the 402,000,000 gals. per day, leaving 383,000,000 gals. per day as the average draft that could be obtained with a storage of 409,000,000,000 gals.

Accepting this assumption, it is evident that 409,000,000,000 gals. is the maximum storage that would be required to prevent any water wasting over the spillway of the New Croton Dam during 40 years, such as those from 1868 to 1907. The average supply that could have been obtained during that time, allowing none to waste, would have been 383,000,000 gals. daily, or 47,000,000 gals. daily more than the supply from the works as they will be at the completion of the Croton Falls Reservoir. This additional storage would probably cost at least $500 per 1,000,000 gals., or a total of $153,000,000 more than has already been spent in the Croton drainage area. This would make the total spent for works to furnish 383,000,000 gals. daily from the Croton drainage area, $241,000,000, not including the distribution system or the proposed filter plant. It is conservatively estimated that not less than 500,000,000 gals. daily can be secured from the Catskill Mountains drainage areas for $162,000,000, including a filtration plant and the main distributing aqueducts to three of the five boroughs.

By means of Fig. 3 there is shown the computed average daily flow of the Croton River, at the New Croton Dam, from 1868 to 1907, with

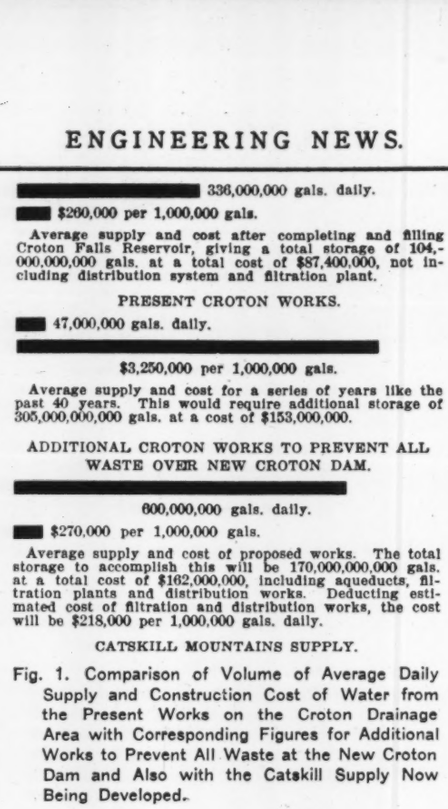

336,000,000 gals. daily.
$260,000 per 1,000,000 gals.
Average supply and cost after completing and filling Croton Falls Reservoir, giving a total storage of 104,000,000,000 gals. at a total cost of $87,400,000, not including distribution system and filtration plant.
PRESENT CROTON WORKS.
47,000,000 gals. daily.
$3,250,000 per 1,000,000 gals.
Average supply and cost for a series of years like the past 40 years. This would require additional storage of 305,000,000,000 gals. at a cost of $153,000,000.
ADDITIONAL CROTON WORKS TO PREVENT ALL WASTE OVER NEW CROTON DAM.
600,000,000 gals. daily.
$270,000 per 1,000,000 gals.
Average supply and cost of proposed works. The total storage to accomplish this will be 170,000,000,000 gals. at a total cost of $162,000,000, including aqueducts, filtration plants and distribution works. Deducting estimated cost of filtration and distribution works, the cost will be $218,000 per 1,000,000 gals. daily.
CATSKILL MOUNTAINS SUPPLY.

Fig. 1. Comparison of Volume of Average Daily Supply and Construction Cost of Water from the Present Works on the Croton Drainage Area with Corresponding Figures for Additional Works to Prevent All Waste at the New Croton Dam and Also with the Catskill Supply Now Being Developed.

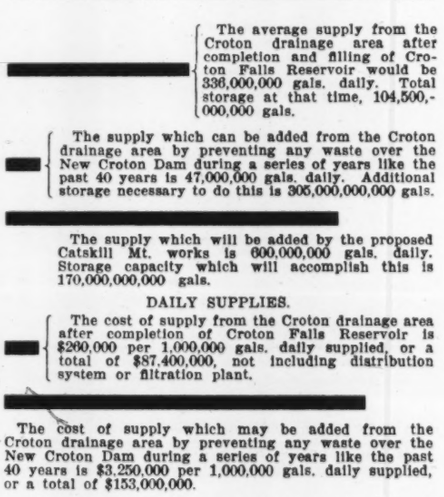

The average supply from the Croton drainage area after completion and filling of Croton Falls Reservoir would be 336,000,000 gals. daily. Total storage at that time, 104,500,000,000 gals.
The supply which can be added from the Croton drainage area by preventing any waste over the New Croton Dam during a series of years like the past 40 years is 47,000,000 gals. daily. Additional storage necessary to do this is 305,000,000,000 gals.
The supply which will be added by the proposed Catskill Mt. works is 600,000,000 gals. daily. Storage capacity which will accomplish this is 170,000,000,000 gals.
DAILY SUPPLIES.
The cost of supply from the Croton drainage area after completion of Croton Falls Reservoir is $260,000 per 1,000,000 gals. daily supplied, or a total of $87,400,000, not including distribution system or filtration plant.
The cost of supply which may be added from the Croton drainage area by preventing any waste over the New Croton Dam during a series of years like the past 40 years is $3,250,000 per 1,000,000 gals. daily supplied, or a total of $153,000,000.
The cost of supply which will be added by the proposed Catskill Mt. works is $270,000 per 1,000,000 gals. daily supplied, or a total of $162,000,000, including distribution mains and filtration plant. Deducting estimated cost of filtration plant and distribution mains, Catskill water will cost $218,000 per 1,000,000 gals. daily.
COSTS PER 1,000,000 GALS. DAILY.

Fig. 2. Comparison of Daily Supplies and Costs of Water from Croton and Catskill Drainage Areas, under Conditions Named in Sub-Titles.

COMPARATIVE YIELDS AND COSTS OF ADDITIONAL STORAGE ON THE CROTON DRAINAGE AREA UNDER VARIOUS CONDITIONS.

	Million gallons.
Total available storage at completion of Croton Falls Reservoir (including flashboard storage)	104,530
Average safe daily supply, assuming no worse conditions than have actually occurred in past 40 years	336
Additional storage necessary to prevent any waste over spillway of new Croton Dam for a 40-year period with runoff conditions as they occurred from 1869 to 1907	305,000
Additional daily supply that would be gained thereby	47
Probable cost of securing this	$153,000,000
Additional storage (above the 104,530,000,000 gals.) necessary to prevent any waste for an 18-year period of low runoff conditions, such as occurred from 1869 to 1886	50,000
Additional daily supply that would be gained thereby	9
Probable cost of securing this	$12,500,000
Additional storage (above the 104,530,000,000 gals.) necessary to prevent any waste for an 18-year period of high runoff conditions, such as occurred from 1887 to 1904	193,000
Additional daily supply that would be gained thereby	103
Probable cost of securing this	$83,000,000

reservoirs as at the completion of the Croton Falls Reservoir, and assuming the evaporation to be from the water surface obtaining with the reservoirs full. It shows the average for the entire 40 years to be 402,000,000 gals. per day; but it also shows that for the 18 consecutive years from 1869 to 1886 the average would be 346,000,000 gals. per day, and for the next 18 years, from 1887 to 1904, on this basis 449,000,000 gals. per day. Taking these two 18-year periods in the same way that we took the 40-year period the results given in the table above were obtained.

Some interesting conclusions were reached by plotting on a piece of tracing paper a line showing the probable future demand for Croton water, beginning at 1908, using the same scales as those of the mass curve. This probable future draft line was placed at various points on the mass curve, and assuming the future runoff to occur exactly as occurred at that time, it was seen at once what the depletion of the reservoirs would be at any time. For instance, starting with all reservoirs full, if the runoff from now on should occur exactly as it did from 1879 onward, the reservoirs now provided for in the Croton drainage area would never again be full until a supply was obtained from some other source. Moreover, by the end of January, 1910, there would be not over 2,000,000,000 gals. left in storage in the entire drainage area, and in October of the same year, the reservoirs would all be entirely empty. Similarly, starting with the runoff as it occurred beginning with the years 1870, 1876, 1880, 1881 and 1891, the reservoirs now provided would never again be full until a supply was obtained from some other source.

It is of interest to note that the entire flow of the Croton River from 1879 to 1886, inclusive, averaged 313,800,000 gals. per day. In other words, the entire runoff for those eight consecutive years averaged about 10,000,000 gals. per day less than the daily average consumption of Croton water for 1907, which was 323,700,000 gals.

While the figure for the average yield for the past 40 years may not be the same as the average for, say, a 60-year period, ending 20 years hence, there is a strong probability that when the total record has covered a long enough period to complete more than one long cycle of wet and dry years, say 80 or 100 years, the daily average will not be far from 400,000,000 gals.

By the above discussion it is clearly demonstrated that, with the completion of the Croton Falls Reservoir, the development of the Croton drainage area will have been carried to the economical limit. The construction of additional reservoirs on the Croton drainage area is not advisable.

All five boroughs of the metropolis need more water; it cannot be gotten too soon. Brooklyn and Queens boroughs could most naturally obtain an additional supply from the water-bearing sands of Suffolk County, Long Island, lying to the east of the city. Richmond Borough (Staten Island), being geographically more closely connected with New Jersey, might naturally look to the adjoining mainland for additional water; but decisions of the New Jersey courts prohibit citizens of that State from supplying water to the island. The local sources are furnishing nearly all they can be depended upon to furnish, and some of them will probably have to be abandoned on account of the deterioration in quality. In an emergency, a temporary small addition to the supply could be obtained from Peekskill Creek and Sprout Brook, north of the Croton drainage area, if permission could be obtained from the Legislature.

It is impossible to place the water supply for the Greater City in a reasonably secure position before 1913, under the most favorable conditions, and the city, as a whole, may be in constant danger of water famine until 1915, or possibly 1920, under adverse circumstances. Making allowance for the possibility that at any time a more severe drought might have occurred than had previously been recorded, it appears that since 1901 New York City has been using more

water than its sources of supply could safely have been called on to furnish as they existed during the past eight years.

With a supply drawn from impounding reservoirs it is essential, especially when dealing with so vast a community, that a margin of safety should be maintained, for if at any time in a protracted dry season the stored water in the reservoirs should be exhausted, the supply would at once be cut down to the dry-weather flow of the stream, a mere fraction of the demand. Furthermore, the probability that reservoirs for impounding the water on new streams can be completed by a certain date does not insure the additional supply at that time, for it may happen that the year succeeding the completion of the new reservoirs may be one of low rainfall, in which case the storage capacity provided could not be fully utilized, and so only a part of the expected additional supply would then become available. Such a condition actually occurred on the Croton drainage area in the spring of 1906, when, although the New Croton Dam was completed, the spring runoff was not sufficient to utilize to any large extent the additional storage provided. The recent, almost uninterrupted, long succession of wet years has masked from the layman, and even from engineers and city officials, the city's real danger. Officials may continue to gamble upon at least moderately wet years and upon the sources of supply continuing to furnish somewhat more water than the engineers calculate to be conservatively safe, and they may depend, for meeting acute emergencies, upon throttling some or all of the distribution mains, at least partially. Such a course, however, is becoming more and more dangerous.

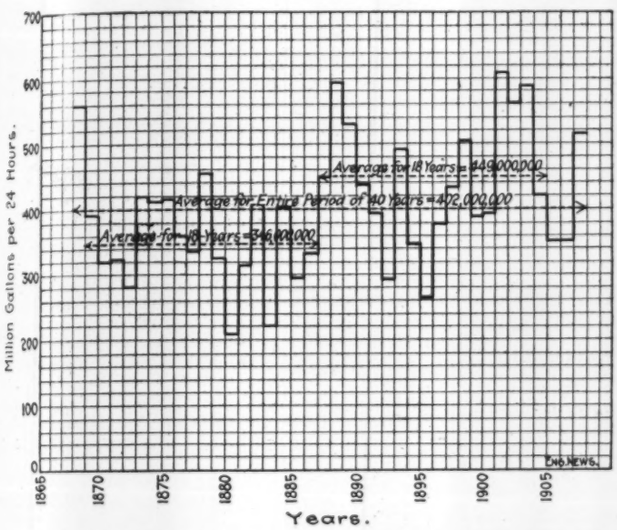


FIG. 3. DAILY AVERAGE RUNOFF OF CROTON DRAINAGE AREA, FOR THE FORTY YEARS 1868 TO 1907, INCLUSIVE.

(Computed natural average daily flow of Croton River at New Croton Dam with rainfall as in each given year, and with reservoir as at the completion of the Croton Falls and Cross River reservoirs. Taken from Table 4, Aqueduct Commission, "Flow of Croton River"; corrected for gain or loss from storage and for evaporation from 5.30% of water surface.)

A CHAPTER HOUSE AT ITHACA, N. Y., occupied by the Alpha Tau Omega fraternity of Cornell University, was completely destroyed by fire on the afternoon of Jan. 31 and one of the student occupants burned to death. The building was of frame construction. Several men were in the house at the time and escaped by the windows and porches, but it was not known that the man who lost his life was in his room. It is thought that he was asleep at the time. The fire recalls the similar tragedy of the near-by Chi Psi house which was destroyed by fire on the night of Dec. 7 with the loss of life of five students and three volunteer firemen. The Alpha Tau Omega house, which was burned on Jan. 31, was situated about half way between the center of the city of Ithaca and the campus of Cornell University, some four hundred odd feet above the city, and on this account was out of the range of the full power of both fire systems. This is the fifth serious fraternity house fire in Ithaca since 1899.

AN ENGLISH AUTOMATIC FIRE-ALARM SYSTEM.

Much inventive energy has been expended in the direction of apparatus for the automatic detection of fires and for giving alarms thereof. Some detectors depend upon the melting of some fusible substance, others upon the expansion of metals when heated. As a rule, fire protection engineers are skeptical towards automatic fire-alarm devices, because in such devices heretofore developed the extreme delicacy of the thermostats, and the uncertain electrical contacts which they make or break, have made them capricious in their operation—i. e., liable to fail at critical times, or to operate unnecessarily and send false alarms.

A fire alarm system noteworthy for its simplicity, was invented and developed by two English engineers some eight years ago, and placed in operation first in New Zealand. After marked success in the British Colonies, and subsequently in large installations in England and Scotland, this system was first demonstrated in this country before the International Association of Fire Engineers at its convention in October. The system has since been exploited in New York City by its inventor, Mr. George H. Oatway,* M. Inst. E. E.

The system, known as the May-Oatway system, and manufactured by the May-Oatway Fire Appliances, Ltd.,† involves, briefly, and in its simplest form, a detector which is placed at the scene of possible fire; a position indicator, connected electrically with the detector; a bell for giving the alarm locally, and an automatic Morse telegraph transmitter of the ordinary type for sending the alarm to the city fire department.

Each detector unit consists of a thin steel channel from 4 to 7½ ft. in length, with 1¼-in. flanges and ¾-in. web. From end to end and within this channel is strung a No. 19 B. W. G. copper wire, slightly slack; the channel is mounted with the open side downwards.

From the center of the wire span is swung a small bob, weighing about an ounce, which forms part of the circuit-closing device shown in Fig. 1. The bottom of this bob is a small cone of carbon, which makes contact—a rubbing contact, absolutely necessary where positive operation is required—when the bob is lowered between the two platinum-faced springs (that is, for open-circuit operation; the apparatus may be so arranged as to break contact, in closed-circuit systems). A glass tube protects the contact surfaces from dust, and permits visual inspection.

The slack on the wire span is regulated by a set-screw at one end, so that the bob is held at any predetermined distance above the springs—the distance depending upon the temperature rise which it is desired shall operate the detector. Any sudden rise in temperature will cause the copper wire to expand and lower the bob. The length of the wire is such that an almost imperceptible linear expansion is magnified to produce a relatively large vertical movement of the bob.

Gradual changes in temperature, such as those of climate or the normal heating of buildings, do not cause the detector to operate; for the steel bar also expands longitudinally, and under slow increase of heat follows the copper wire sufficiently to compensate for the expansion of the latter. In a sudden rise, such as that caused by a fire, the steel bar does not expand rapidly enough to catch up with the fire, and the detector operates.

Detectors are placed wherever protection is necessary; swung horizontally near the ceiling, and each detector is connected with the centrally located position-indicator case. The wiring of the essential part of one complete circuit is shown in accompanying diagram, Fig. 2.

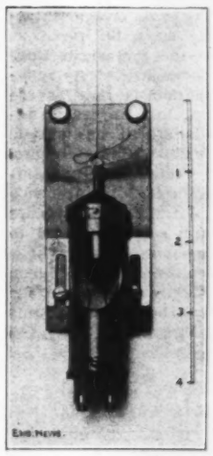


Fig. 1. Contact-Making Portion of May-Oatway Fire Detector.

This is mounted on one flange of the channel bar at the center. The relation between the conical carbon button and the contact springs should be noted, as it produces a rubbing contact.

An ordinary gravity drop and two relays are connected in series with the detector and a 10-volt battery. The drop indicates the location of the fire, one relay operates a magnetic brake-release, which starts the Morse transmitter, and the other relay closes the local alarm-bell circuit. The two relays, the bell and the transmitter, are of course, common to all detector circuits. All of this apparatus is located in a convenient wall case, supplemented with the necessary testing appliances, switches, etc.

The transmitting wheel of the Morse instrument is so arranged as to send a fixed code signal repeated four times. Only the receipt of four such signals will call the fire department into action, as only the heat of a fire will keep the transmitter in operation through its full period. A contact at the detector caused by jarring, or by testing, will not be of sufficient duration to send the complete signal—hence, unnecessary trips by the fire department are avoided.

Where the system is to be employed merely to give alarms of fire to the inmates of a building the telegraph instrument is, of course, omitted. Under such conditions the number of alarm bells may, of course, be increased to as many as may be desired.

As to rapidity of operation, sundry tests made in the presence of a representative of Engineering News proved the system to be remarkably prompt. In a temporary installation with two detectors, in a room with a ceiling about 10 ft. high, the detectors being swung at the ceiling, three trials were made by kindling fires on the floor in different parts of the room. The room temperature was 78° F.

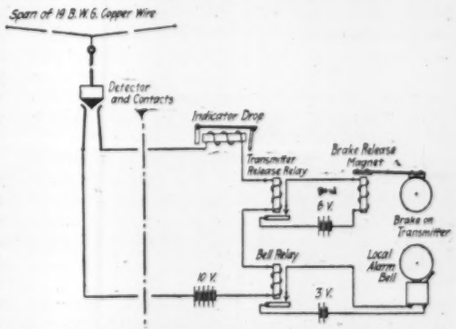


Fig. 2. Arrangement of One Complete Circuit, May-Oatway Automatic Fire Alarm System.

In the first test, one detector 30 ft. from the fire, the draft being towards it, operated in 24 seconds; the other detector, 15 ft. from the fire and "to windward" of it, operated in 1 min. 20 sec. In the second test, made only with ref-

*82 Milton Place, Highgate, London, N.
†92-94 Paul St., London, E. C.

erence to the second detector, previously mentioned, the detector operated in 23 seconds 10 ft. from the fire, with no draft in the room. In the third test, by which time the temperature in the top of the room had reached 82° F. from the two previous fires, the same detector operated in 10 seconds with a fire 4 ft. from the wall on which it was swung. The fire in each case was a large pan of burning alcohol, and both detectors were set to operate on a 10° rise.

Practical installations of the system in England and Scotland have demonstrated its reliability. The largest installation is that at the Anchor Mills of Clark & Co., Ltd., thread manufacturers, at Paisley, Scotland. In this system there are 119 miles of internal alarm circuits, 5¼ miles of underground cable between buildings, 19 automatic transmitters, 17 firemen's telephones and 20 12-in. alarm gongs. Besides this, an automatic telegraph set is provided to call the Paisley fire department.

For the information, diagrams, etc., from which this article was prepared we are indebted to Mr. Oatway. The apparatus will be handled in this country by the Gamewell Fire-Alarm Telegraph Co., of New York City.*

SALARIES OF TECHNICAL GRADUATES.

By WALTER B. SNOW,† M. Am. Soc. M. E.

While we may deplore the ready acceptance of earning power as a measure of attainment, nevertheless the individual salary furnishes the simplest approximate measure of what the world chooses to call success. In recent discussions of the value of technical training, this standard of comparison has been readily accepted. Although it has been recognized that technical graduates are really picked men, who would in many cases have risen above the average, yet the monetary value of their education is self-evident.

During the early years after graduation, there are likely to be great differences in income. These differences primarily result from the fact that some men, either as graduate students, engineering apprentices, or the like, receive little or nothing during this period, while others, through various influences, jump into exceedingly remunerative positions. But as time passes these differences decrease; the true work of the individual becomes manifest, and except for the influence of sporadic extremes, the average salary of a considerable group becomes a fair token of worldly success.

Some interesting facts of this character appear in a recent class publication of one of our leading technical schools. The record was made ten years after graduation, confidential information regarding salaries having been secured from 151 members of the class, of whom about 75% were graduates, the balance having been special students. The majority of the graduates were from courses in civil, mechanical, and electrical engineering, chemistry, and architecture; nearly one-third of those comprised in this classification having completed the course in electrical engineering.

Comparison of salaries was made between limits, rather than in specific sums. The results are shown by the accompanying diagram, which presents the percentage of the total number heard from, who, ten years after graduation, were receiving salaries ranging between the amounts indicated. In gross amounts:

113 graduates receive a total of	$334,334 annually.
38 specials receive a total of	134,848 annually.
151 or total number reported received	$469,182 annually.

*19 Barclay St.
†Publicity Engineer, 170 Summer St., Boston, Mass.

The smallest salary is $728, while the highest paid man receives $20,000.

Further analysis shows that

The average salary of the graduates is	$2,958
The average salary of the specials is	$3,548
The average salary of the class is	$3,107
The salary of the 76th or medium man is	$2,500

The fact that the average of the specials exceeds that of the graduates may be misleading unless it be remembered that some specials, due to exceptional influences, are receiving very large salaries, and furthermore that although not having completed the prescribed course, they may have been in business more than ten years. By eliminating the two lowest and the two highest salaries, a fair average of $2,942 is obtained. Again, by considering the medium man's salary, $2,500, as the average, it appears that 49% are below that average, and 45% above, while six have the average salary.

The same report gives an interesting comparison with the records of the three classes preceding. From this it appears that the percentage receiving $1,500 to $1,999 in the four classes, ten years after graduation, were respectively 23.2, 25.2, 18.9 and 19.8; while those receiving $2,000 to $2,499 represented 16.8, 13.7, 23.4 and 17.9% of the entire class in each case. Obviously these records refer respectively to the years 1904, 1905, 1906 and 1907; this fact may have had some influence in determining the proportions.

In a general way the results speak for themselves. The positions held vary greatly in their character. A large majority of the men have not yet attained marked responsibility; many hold the position of assistant in the various industries, or the equivalent in purely professional lines. As the average age of these men is probably about thirty-two years, there is still ample time for advancement. The immediate future should show a marked increase in many salaries, as the men, to a considerable extent, stand on the thresholds of positions of command and primary importance.

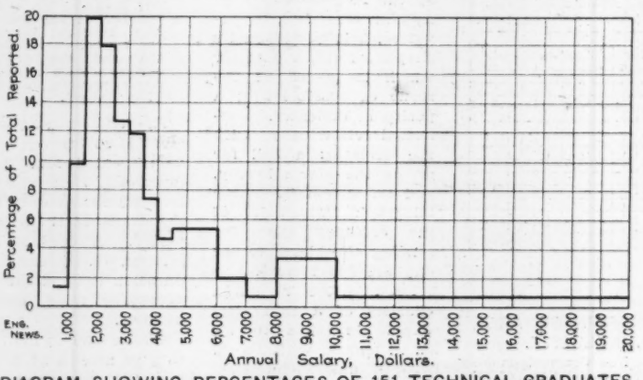


DIAGRAM SHOWING PERCENTAGES OF 151 TECHNICAL GRADUATES OR SPECIALS WHO WERE RECEIVING CERTAIN SPECIFIED SALARIES TEN YEARS AFTER GRADUATION.

At the same time income from investment, or from direct interest in the industry with which they are associated, will doubtless add much to the merely nominal salaries received by many. It should be noted that salaries alone have been considered in this analysis.

Could comparison be based upon the actual results accomplished, upon the additions to the world's knowledge, and upon the advance in the welfare of mankind, the measure of success would doubtless far exceed that attained by recipients of like salaries in commercial life. But the technically trained man must still remain content with the lesser income, and accept as partial recompense the solid satisfaction of having advanced the material conditions of the world in which we live.

NOTES FROM ENGINEERING SCHOOLS.

MASSACHUSETTS INSTITUTE OF TECHNOLOGY.—A seminar is to be maintained during the year 1908-9 for studying the trend of contemporary advance in the electrical arts and its relation to historical development. This is a departure from the customs of engineering schools, but has been judged desirable for men following an advanced course of electrical engineering. It is included as a necessary part of the training of proper judgment of those who hope to hold executive positions of commanies depending on the electrical arts.

A course will be maintained for graduate students in the organization and management of public service companies. This course will include lectures on the methods of organization, legitimate expenses, problems of management and public relations from an engineering standpoint. Public relations will also be treated by a course of lectures, under the department of economics, from the viewpoint of the political economist. This course will be accompanied by extensive assigned reading and by examinations and comparisons of operating reports, which may be accompanied by research of a historical or economic nature in the case of students suitably prepared.

STEVENS INSTITUTE OF TECHNOLOGY.—The seventh annual dinner of the Alumni Association of Stevens Institute was held at the Hotel Astor, New York City, Feb. 4, with about 250 graduates and guests present. Among the speakers were President A. C. Humphreys, Dr. H. S. Prichett, President of the Carnegie Foundation for the Advancement of Teaching; Hon. R. P. Porter, former Chief of the Census Bureau; Mr. J. E. Sague, New York Public Service Commission, and many others.

President Humphreys, in his address, advocated the inclusion in the curriculum of a technical school, of a course in the economics of administration and the development of courses in respect to the business side of engineering. He appealed for the greatest thoroughness in teaching the fundamentals of engineering science and in developing the ability to analyze data and to logically reason rather than to memorize formulas and figures. President Humphreys took occasion to champion the cause of Andrew Carnegie in the recent attack of Mr. R. T. Crane, before the Western Ry. Club, on technical and industrial education and on Mr. Carnegie for supporting engineering education.

ANNUAL MEETING OF THE CANADIAN SOCIETY OF CIVIL ENGINEERS.

The annual meeting of the Canadian Society of Civil Engineers, held at Montreal last week, was a well attended and enthusiastic gathering; over 10% of the society's members were registered at the convention, and the society's house was taxed to accommodate the assemblage.

We present herewith a group photograph of a few of the members in attendance, for which we are indebted to the courtesy of Mr. Jos. W. Heckman, Assoc. M. Can. Soc. C. E. The photograph gives a graphic idea of the weather conditions. The mercury ranged down to 20 degrees below zero. The snow was knee-deep and the snowbanks in the streets shoulder high; but—to paraphrase Kipling—they don't call this cold in Montreal; and the weather had no effect on the success of the convention.

At the meeting on Jan. 28, the Council's report was presented showing that the society had gained over 400 new members during the past year—a growth which breaks all records. The officers elected for the coming year were: President, Dr. J. Galbraith, Toronto, whose portrait and biography appear elsewhere in this issue; Vice-Presidents: W. F. Pye, Montreal; M. H. MacLeod, Winnipeg; and C. H. Duggan, Sydney.

The address of the retiring president, Mr. W. McLea Walbank, of Montreal, was devoted to the subject of municipal ownership. Mr. Walbank opposed municipal ownership and declared that the experience of Glasgow and other British cities was not a safe guide for cities on this side of the water. Mr. Walbank also opposed the idea that competition should be relied upon to regulate the rates and service of public utility companies and protect the public. The best remedy for the evils of monopoly was public control, and he favored a public service commission

organized on similar lines to the present Canadian Railway Commission.

At the annual business meeting the further legislative protection of the profession was discussed at length. It was thought that the Quebec act should be more strictly enforced; but in Ontario the opinion was expressed that it was hopeless to expect favorable legislation.

In a discussion regarding the engineering status there was a general consensus of opinion that the time is approaching when engineers seeking commission, who are not graduates of engineering schools, should be required to undergo an examination similar to that demanded by the Institution of Civil Engineers of Great Britain.

A notable feature of the convention was the exhibit of lantern slides of the Quebec Bridge

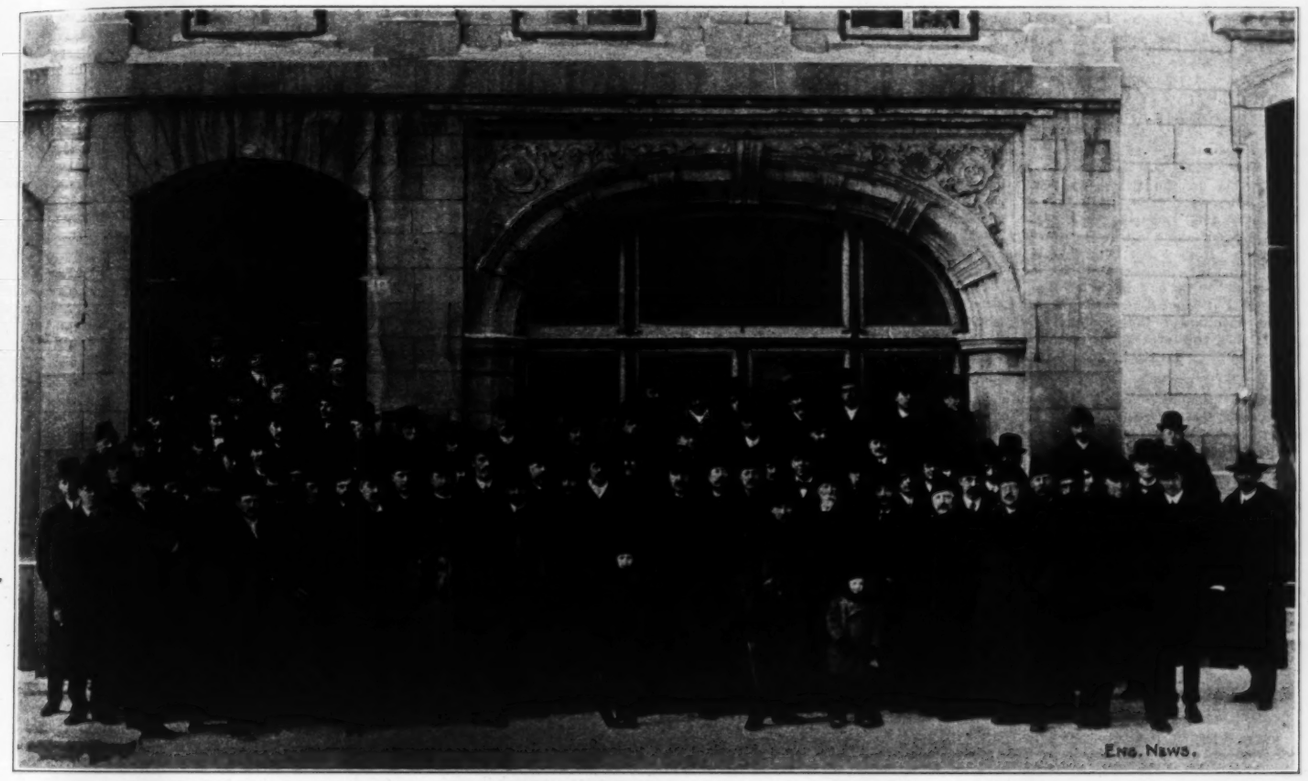
A COAL-DUST MINE EXPLOSION in one of the mines of the Boone Coal & Coke Co., near Hawksnest, W. Va., on Jan. 30, killed at least nine miners and injured the other four or five men who were in the mine at the time. The mine is a small one working at the maximum output only 25 men.

AN EXPLOSION IN THE SHRAPNEL SHOP of the Franklin Arsenal at Philadelphia, on Jan. 31, injured eight men, three of them so seriously that they may not recover. So far as can be discovered the explosion was caused by the ignition of a ten-grain primer, the flames from which spread to several hundred others. The entire upper part of the building was wrecked.

THE STEAMER "ST. CUTHBERT," en route from Antwerp to New York, was burned to the water's edge off the coast of Nova Scotia on Feb. 2. Fifteen members of the crew were drowned by the swamping of a small boat in which they attempted to leave the vessel. The other

"rolling-lift" bascule bridge across a channel 65 ft. wide between abutments. It is being built for the Department of Bridges of New York City by the Godwin Construction Co.; the steel work of the draw is being built and erected by the American Bridge Co. on a sub-contract. The two leaves were being erected in raised position, and as the counterweights were not yet in place they were held up by cable guys. Both leaves were nearly complete. High winds prevailing at the time of the accident may have torn one of the guys from its fastenings. The southerly leaf of the draw fell forward into the channel and jumped off its track in falling. Mr. J. G. Theban is Resident Engineer in charge of the construction of the bridge for the city.

BIDS FOR A DIRIGIBLE BALLOON for the War Department have been readvertised. The date of opening has been fixed as Feb. 15. The specifications are very broad, dimensions and shape being left to the bidder except that the length must not exceed 120 ft. The



GROUP IN FRONT OF THE HOUSE OF THE CANADIAN SOCIETY OF CIVIL ENGINEERS IN MONTREAL AT THE SOCIETY'S ANNUAL MEETING, JAN. 28-30.

given by Mr. Henry Holgate, Chairman of the Commission entrusted with the investigation of the Quebec Bridge disaster.

The various excursions laid down in the program, printed in our issue of Jan. 23, were well attended and the annual dinner at the Windsor Hotel on Wednesday evening was a most enjoyable affair.

A TORNADO swept over a distance of 40 miles through the southern part of Mississippi on Jan. 31, destroying considerable property and causing the death of at least 20 persons. Fortunately there were no towns of any size in its path.

THE WORST RAILWAY WRECK OF THE WEEK occurred on Feb. 2, about 47 miles southwest of Detroit, Mich., when a Wabash R. R. express en route from St. Louis to Detroit was thrown from the track and overturned by a broken rail. One passenger was killed and 34 injured.

A BOILER EXPLOSION in the rolling mill of Van Allen & Co., Northumberland, Pa., killed seven men on Feb. 3, and destroyed about $75,000 worth of property. The explosion occurred as the men were just preparing to resume work after a three months' shut-down, during which time the machinery had been overhauled.

37 members of the crew were rescued by the White Star liner "Cymric," which, in a terrific blizzard, stood by the burning vessel for nine hours. The "St. Cuthbert" was owned by the British & Foreign Steamship Co., of Liverpool, and was a freighter of about 5,000 tons register. The floating hulk was abandoned and is now a dangerous derelict in the path of the North Atlantic steamers.

A WIRE ROPE TRAMWAY used to handle spoil and material in the construction of the new 73d St. water intake tunnel at Chicago, Ill., was wrecked by ice on Jan. 30. The tramway extended from the shore to a construction crib about 1½ miles out, and consisted of 26 steel towers, 300 ft. apart, about 35 ft. high above water, carrying the track cable, etc. It was described in our issue of Nov. 14, 1907, p. 511, in an article "New Water Supply Tunnels at Chicago." The tramway was built by J. W. Jackson, Inc., who is contractor for the intake tunnel and crib. On Jan. 30 there was a heavy ice shove in the lake, and the towers of about two-thirds of the tramway were wrecked. Work will be delayed by the accident, but Mr. Jackson is quoted as saying that the tramway will be rebuilt in a month.

A BASCULE BRIDGE FELL DURING ERECTION at the crossing of Eastern Boulevard over Eastchester Creek, Bronx Borough, New York City, on Sunday, Feb. 2. The wrecked structure is in the draw span of a bridge some 800 ft. long, consisting of six 105-ft. reinforced-concrete arch spans and a two-leaf Scherzer

opening of the first bids for this dirigible balloon was noted in Engineering News, Jan. 23.

THE HUDSON RIVER TUNNEL, between Morton St., New York City, and Hoboken, N. J., is to be opened for traffic on Feb. 25, according to present plans of the owners, the Hudson Companies. The New York extension of the tunnel will be operated as far as Sixth Ave. and 19th St. for the present, pending completion of the remainder of the extension to Sixth Ave. and 33d St. and the branch along Ninth St. to Fourth Ave.

THE IROQUOIS THEATRE DISASTER was the subject of an important legal decision in Chicago, Ill., on Feb. 1. Judge Chytraus held that the builders of the theater were not absolved by the Building Commissioner's approval of their plans from liability for violation of the building ordinances. The builders had been sued for damages by relatives of a number of victims of the disastrous fire, and although a prior judge had held that the Building Commissioner's approval freed the builders from liability, the present decision takes the opposite view.

PANAMA CANAL WORK DURING JANUARY, just reported by cable to Washington, maintains the increasing rate of output. The total excavation for the month was 2,712,568 cu. yds. place measurement, against 2,200,539 cu. yds. in December and 1,838,486 cu. yds. in November. The average daily output in January, 26

working days, was 104,330 cu. yds., against 88,000 per day in December with 25 working days. The rainfall was 1.15 ins. against 3.85 ins. in December. The January excavation is distributed as follows, the December figures being also given. (See Eng. News of Jan 9, 1908, p. 49, for the December and November figures).

	January, 1908. cu. yds.	December, 1907. cu. yds.
Steam Shovels:		
Culebra Division	1,150,334	1,006,026
Gatun Locks and Spillway	193,567	160,795
Mindi, Chagres and Miraflores	239,485	152,117
Dredges:		
Colon Division	444,403	428,053
La Boca Division	460,250	349,551
Total, Canal prism	2,488,039	2,096,542
Accessory works outside the canal prism	224,529	103,997
Total Excavation	2,712,568	2,200,539

This brings the total excavation during the American occupation to 25,461,036 cu. yds. at the end of January, 1908. Of this total, steam shovel work within the canal prism constitutes 16,140,967 cu. yds.; dredging within the canal prism accounts for 4,431,664 cu. yds., and excavation outside the canal prism (mostly dredging), 4,888,405 cu. yds.

THE THIRD RAILWAY ACROSS CENTRAL AMERICA, the Guatemala Northern Ry., was opened on Jan. 19, 1908, in the presence of President Cabrera and other officials of the government and an American delegation headed by Maj.-Gen. Geo. W. Davis. The railway connects Puerto Barrios on the Caribbean Sea with Guatemala City, 195 miles to the south, in the mountains on the Pacific slope, and connects there with the Guatemala Southern Ry., running 75 miles to San José on the Pacific coast. It is worth noting that just a year ago, Jan. 23, 1907, the Tehuantepec National Railroad was opened, as noted in our issue of Jan. 31, 1907. This was the first transcontinental line in Latin America since the opening of the Panama Railroad, and the Guatemala line now makes the third.

SHORTAGE IN FIRE HOSE in New York City is declared by the Commissioners of Accounts to be very serious. They declare, as one result of an examination of the Fire Department in connection with the destruction of the Parker Bldg. (Eng. News, Jan. 16, 1908, p. 73), that about 4,000 lengths of hose (200,000 ft.) are needed at once, as about one-third of the amount of hose now in the department is unfit for use, largely because it is too old. The new high-pressure fire service in the downtown district, soon to be put in operation, will call for specially strong hose, and one-third of the new hose should be high-pressure hose (250 lbs.). As regards the Parker Building fire, however, the Commissioners conclude that most of the blame for the large extent of damage falls on the Acting Chief who was in charge, his engines having been improperly placed to get sufficient water from the available water mains.

PERSONALS.

Mr. Parker Dodge has connected himself with the Pittsburg office of the Youngstown Car Manufacturing Co.

Mr. H. W. Nutt has accepted a position as District Manager for the New England States for the General Fireproofing Co., with headquarters at Boston.

Mr. Lindon W. Bates has resigned as President and Director of the Empire Engineering Corporation and Mr. William Barclay Parsons has been elected to succeed him. Mr. Parsons has always been one of the board of directors of the company.

Mr. Frank Cooper, formerly a civil engineer in the Pennsylvania R. R., the Norfolk & Western Ry. and the Baltimore & Ohio R. R., has been admitted to the bar of the Supreme Court of Pennsylvania and will soon open an attorney's office at Somerset, Pa., for the prosecution of a law business along engineering lines.

Mr. George L. Noble, who has been for the past 20 years in the continuous service of the Gould lines in Texas, has resigned as Assistant General Manager of the International & Great Northern R. R. His successor is Mr. Henry Martin, formerly Superintendent of the Fort Worth Division of the same road.

Major John Stephen Sewell, Corps of Engineers, U. S. A., has resigned from the army and entered into the private practice of his profession. Maj. Sewell was graduated from the Military Academy in 1891. His principal assignments in the government service were the construction of the new Government Printing Office and of the War College, both at Washington, D. C.

Mr. John A. Bensel, M. Am. Soc. C. E., has been appointed a Commissioner of the Board of Water Supply of New York City and elected chairman of the body by the other two members. Mr. Bensel has served the city for the past ten years in the Dock Department, from 1898-1906 as Chief Engineer and since 1906 as Dock Commissioner. He is a graduate of Stevens Institute and has been professionally connected with the Aqueduct Commission of New York, the Pennsylvania R. R., and the Philadelphia Water Dept. No successor to the office of Dock Commissioner has yet been appointed.

Mr. David E. Whitford, of the Syracuse office of the State Engineer of New York, was tendered a dinner on Jan. 24, by his associates, in honor of his long service in the department. Mr. Whitford entered the office of the State Engineer in 1852 and has spent all but about five years' of the time since then in continuous active duty. He has served under every State Engineer since the formation of the office in 1848, except the first two, and has held office from tapeman to division engineer. For 14 years he was in charge of the Cayuga & Seneca, the Chemung and the Crooked Lake Canals, but the larger part of his time has been spent in the city of Syracuse. Mr. Whitford is now in his 79th year and is daily at work in the Division Engineer's office at Syracuse.

Obituary.

Charles B. Elliott, who was the first general electrical engineer of the Pullman Palace Car Co. and later in the car lighting department of the Pennsylvania R. R., died in Jersey City on Feb. 1.

Sir Henry W. Tyler, from 1877 to 1895 President of the Grand Trunk Ry. of Canada, and more recently the Chairman of the Westinghouse Brake Co. of England, and Deputy Chairman of the Great Eastern Ry. Co. of England, died at his home in London on Jan. 30 at the age of 81 years. He was knighted in 1877 and was a member of the British Parliament for a considerable time.

Thomas Mellon, one of the original Pittsburg steel men, died at his home in that city on Feb. 3 at the age of 95 years. Mr. Mellon was mainly connected with the large financial institutions of Pittsburg, but was also the founder of the Union Steel Co., the Aluminum Co. of America, the Carborundum Co. of America and many other large manufacturing concerns.

Rafael Guastavino, the inventor of the Guastavino tile system of floor construction and the president of the company which has exploited it, died at his country place, near Asheville, N. C., on Feb. 2, at the age of 55 years. The system with which Mr. Guastavino was identified was one of the first fireproof floor systems put into use in this country, and has been described quite fully in many of the early issues of Engineering News. It consisted of a flat tile arch or dome spanning between floor beams spaced farther apart than is now customary in similar work. The most notable building in which it was used was the Boston Public Library.

Othniel F. Nichols, M. Am. Soc. C. E., formerly Chief Engineer of the Department of Bridges of New York City, died suddenly at his home in Brooklyn, N. Y., on Feb. 4. Mr. Nichols was born in Newport, R. I., July 29, 1845, and received his technical education at the Rensselaer Polytechnic Institute, graduating therefrom in 1868. His first professional work was in connection with the laying out of Prospect Park in Brooklyn. From 1871-75 he was in Peru, engaged in the construction of the Chimbote R. R. Upon his return from South America Mr. Nichols became an assistant engineer in the Edgemoor Bridge Works and later engineer in charge of a large portion of the sewerage work of the city of New York. In the year 1878 he again went to South America as resident engineer on the noted Maderia & Mamoré R. R. project in Brazil. He was connected with the construction of the various street railways of New York City and served as Chief Engineer and also as General Manager of the Brooklyn Elevated Ry. from 1888 to 1895. In 1896 he became connected with the Department of Bridges of New York and immediately assumed charge of the construction of the Williamsburg suspension bridge across the East River. In 1903 he was appointed Chief Engineer of the Department, holding that position until 1906, when he was succeeded by C. M. Ingersoll. Since 1906 Mr. Nichols had been a consulting engineer of the department. He was a member of the American Society of Civil Engineers, the American Society of Mechanical Engineers, the Institution of Civil Engineers and many other technical, educational and social organizations. He is survived by a widow and two daughters.

ENGINEERING SOCIETIES.

COMING MEETINGS.

NATIONAL BRICK MANUFACTURERS' ASSOCIATION.
Feb. 3-8. Annual meeting at Columbus, Ohio. Secy., Theo. A. Randall, Indianapolis, Ind.

AMERICAN SOCIETY OF PLUMBING INSPECTORS AND SANITARY ENGINEERS.
Feb. 10-12, 1908. Annual meeting at Chicago, Ill. Secy., Garrett W. Cotter, 44 East 23d St., New York City.

CONNECTICUT SOCIETY OF CIVIL ENGINEERS.
Feb. 11-12. Annual meeting at New Haven. Secy., T. F. Jackson, New Haven.

OHIO ENGINEERING SOCIETY.
Feb. 11-13. Annual meeting at Columbus. Secy., E. G. Bradbury, 85 North High St., Columbus.

AMERICAN INSTITUTE OF MINING ENGINEERS.
Feb. 18. Annual meeting at New York. Secy., R. W. Raymond, 29 West 39th St., New York.

AMERICAN RAILWAY ENGINEERING AND MAINTENANCE OF WAY ASSOCIATION.
March 17-19, 1908. Annual convention at Chicago, Ill. Secy., L. C. Fritch, 1562 Monadnock Block, Chicago.

NEW YORK RAILROAD CLUB.—A dinner commemorative of the long and distinguished services of Mr. H. H. Vreeland as president of the club, will be given on Feb. 7.

AMERICAN RAILWAY ENGINEERING AND MAINTENANCE-OF-WAY ASSOCIATION.—The following list of nominations for officers has been issued: President, Walter G. Berg (Lehigh Valley R. R.); Vice-President, L. C. Fritch (Illinois Central R. R.); Director, C. S. Churchill (Norfolk & Western R. R.) and E. Wendt (Pittsburg & Lake Erie Ry.); Treasurer, W. S. Dawley (Mo. & No. Arkansas Ry); Secretary, E. H. Fritch.

CONNECTICUT SOCIETY OF CIVIL ENGINEERS.—The principal papers to be delivered at the annual meeting of the society at New Haven, Feb. 11-12, are as follows: "The Purification of Water by Sand Filtration," W. B. Fuller; "Reinforced-Concrete Bridges," M. W. Denman; "The Harbor and Rivers of New Haven," J. F. Jackson; "Stream-Flow Data," C. E. Chandler; "Water Waste Investigations," W. E. Johnson; "Hydraulic Fill Dams," W. S. Morton.

OHIO ENGINEERING SOCIETY.—At the annual meeting of the society at Columbus, Feb. 11-13, the following are among the more important papers to be delivered: "Sewers and Sewer Methods," W. H. W. Jenkins; "Reinforced-Concrete Bridges," F. A. Bone; "Explosions in Coal Mines," Andrew Roy; "Cement Sidewalk Construction," L. A. Keith; "Lining a Broken Reservoir with Reinforced Concrete," Wm. Wilson; "City Wastes Disposal and Street Cleaning," Paul Hansen.

AMERICAN INSTITUTE OF ELECTRICAL ENGINEERS.—The annual dinner will be given in the grand ballroom of the Waldorf-Astoria, New York City, Wednesday, Feb. 19, at 7 P. M. Following the custom of previous years the after-dinner speaking will all be on a special topic. The dinner in 1903 was called the "Library Dinner." In 1904 the "Edison Dinner" commemorated the introduction and development of the incandescent lamp. In 1905 this function was called the "Traction Dinner." The present occasion will be known as the "Public Service Dinner," and among the speakers who have promised responses to toasts are many men prominently identified with public service utilities, either as members of commissions or operating heads of large utilities' organizations. The price of the dinner will be $5, not including wine or cigars. As usual, ladies will be present. Guests will be seated at small tables accommodating eight persons. The dinner committee is composed of Robert T. Lozier, Chairman; A. A. Gray, Frederick C. Bates and George H. Guy.

AMERICAN INSTITUTE OF MINING ENGINEERS.—The annual meeting will be held at the headquarters of the institute, in the United Engineering Societies Building, New York, on Feb. 18 to 21. At the evening meeting on Feb. 18 there will be a paper on "Electric Power in Steel Mills," by D. B. Rushmore. At the morning session on Feb. 19 there will be a special discussion on "The Physics of Steel," being a continuation of former discussions on the subject. It is intended to relate specially to the structure of steel ingots and their mechanical treatment in the production of forgings, rails, beams, etc. There will also be further discussion of Prof. H. M. Howe's paper on "Piping and Segregation in Steel Ingots," and a paper by Mr. J. E. Howard on the work of the testing department at Watertown Arsenal and its relation to metallurgical matters. On Feb. 20 there will be a reception, and on that day and Feb. 21 excursions will be arranged to the East River tunnels, the Tunnel Terminal Building, the Standard Oil refinery at Bayonne, and the Astoria plant of the Consolidated Gas Co. A number of papers and discussions are on the program for the first two days.

WESTERN SOCIETY OF ENGINEERS.—The following program has been prepared for the present season, meetings being held at the society's rooms in Chicago: Feb. 5—Test of a small suction gas producer plant; H. B. MacFarland. Feb. 14—The evolution of the electric railway; J. N. Hatch. Feb. 19—The passing of the electric locomotive; W. E. Symons. March 4—The location, arrangement and construction of manufacturing plants; George M. Brill. March 13—Alternators in parallel; Prof. Morgan Brooks. March 18—Recent development in the steam turbine; Prof. J. C. Thorp. April 1—The Construction of the South Side Elevated Ry.; J. N. Darling. April 10—The relation between the banker and the engineer; J. C. Kelsey. April 15—Notes and specifications on macadam road construction; A. N. Johnson. May 6—A general discussion on the desirability (from a financial standpoint) of using ballasted bridge floors and of continuing the use of ordinary pile bridges with open floors. May 8—The electrical equipment of steam and gasoline automobile; F. R. Babcock. May 20—Economics of grade reductions in railway operations; W. D. Taylor. June 3—The increase of the physical strength of a paint coating for the protection of structural surfaces; R. S. Perry.